UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

HYDRAULIC MODEL STUDIES OF BOYSEN DAM SPILLWAY

Hydraulic Laboratory Report No. 212

ENGINEERING LABORATORIES BRANCH

DESIGN AND CONSTRUCTION DIVISION
DENVER, COLORADO

January 8, 1952

FOREWORD

Hydraulic model studies were conducted on two separate schemes of Boysen Dam spillway at two different times. Studies made of Scheme Two are discussed first in this report, while studies made of Scheme One are considered secondly.

Hydraulic model studies of Scheme Two were conducted in the Hydraulic Laboratory of the Bureau of Reclamation at the Denver Federal Center, Denver, Colorado, during the period May 27, 1947 to June 7, 1949. The results obtained were brought about through the cooperation of the staffs of the Spillway and Outlet Section No. 1 and the Hydraulic Laboratory. During the course of the model studies, Messrs. D. C. McConaughy and L. M. Stimson of the Spillway and Outlet Section No. 1 frequently visited the laboratory to observe the model tests and to discuss the test results. These studies were conducted by Messrs. G. L. Beichley and W. E. Wagner under the direct supervision of Messrs. A. J. Peterka and J. N. Bradley.

Hydraulic model studies of Scheme One of Boysen Dam spillway, Section II of this report, were conducted in the Hydraulic Laboratory of the Bureau of Reclamation at the United States Customhouse, Denver, Colorado, during the period January 1946 to November 1946. These studies were conducted by Mr. R. C. Besel under the direct supervision of Mr. J. N. Bradley and in cooperation with Messrs. D. C. McConaughy and D. A. Dedel of the Spillway Design Section.

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Hydraulic Laboratory
Written by: G. L. Beichley
Reviewed by: A. J. Peterka

Subject: Hydraulic model studies of Boysen Dam Spillway

Scheme Two

SUMMARY

Hydraulic model studies of Boysen Dam spillway, Figures 1 to 4, inclusive, conducted on a 1:48 scale model, Figures 5 to 9, inclusive, for the purpose of developing and checking the hydraulic design by means of performance tests indicated that the structure in general was satisfactory. Data and notes taken on the flow in the spillway approach, over the spillway crest, in the stilling basin, and in the river channel immediately downstream showed the design of the stilling basin to be the major problem.

Performance tests on seven different stilling basins, Figure 17, were made to determine the best arrangement of baffle piers and sills and to determine the most economical training wall along the right side of the basin that would protect the area behind the powerhouse from waves and splash originating in the stilling basin. On each of the seven stilling basins, tests were conducted to determine the design that would give the smoothest water surface, Figures 18, 19, 22(a), 23(a), 24(a), 25(a), 26, 27, 29, and 30; lowest water-surface profile, Figure 20; the least scour near the end of the apron, Figures 21, 22(b), 23(b), 24(b), 25(b), 28, and 31; the most stable hydraulic jump; and the least objectionable eddy in the flow pattern downstream and to the left of the stilling basin. A summary of the test data on all seven stilling basin designs is recorded in Figure 32.

Two stilling basin designs were found from the tests to provide satisfactory performance. The basin adopted by the designers used the preliminary stilling basin design, Figure 17, and a modified right training wall. The preliminary design was satisfactory for flows not exceeding 20,000 second-feet except that tests showed the preliminary right training wall of the stilling basin, which joined the left wall of the powerhouse, to be too low, as it was overtopped by waves originating in the stilling basin. Various methods to prevent the turbulent flow from overtopping the wall were tested, but

the most satisfactory and economical method was to extend the wall 3-1/2 feet in height, turning the top inward 3 feet, as shown in Section G-G of Figure 3.

The stilling basin recommended by the laboratory is shown in Figure 17. It was similar to the preliminary design, except that the dentated sill at end of the basin was located 24 feet farther downstream which, in effect, lengthened the stilling basin. The performance of this design shown in Figures 29 and 30 compared to the performance of the preliminary design shown in Figures 18 and 19 gave a smoother water surface in and below the basin together with better all-around performance. For the recommended design, the high point of the water-surface profile occurred along the high powerhouse wall, instead of along the low training wall upstream, Figure 20. The jump was found to be stable for a greater range of tail-water elevations than for any other design tested, Figure 32. Scour tests results for the recommended design were found to be similar to those of the preliminary design, as can be seen by comparing Figures 21 and 31.

Tests were conducted in the approach to the spillway to determine the effect on spillway capacity and on flow patterns caused by the relocation of the powerhouse intake and trashrack structure to a more favorable foundation, Figure 9. Relocation of the trashrack proved to increase the spillway capacity slightly, Figure 11, and decrease the surface disturbance of the flow entering the right spillway bay, Figures 10 and 12.

With the trashrack in its final location, crest pressures were checked and extensive discharge calibration tests performed. All pressures recorded on the spillway crest were found to be above atmospheric, except for a slight subatmospheric pressure just downstream from the crest near Station 6+06.3 for 20,000 second feet with maximum reservoir surface, Figure 13. Discharge calibration tests were conducted to determine the gate settings required in the prototype to maintain the design discharge of 20,000 second feet, Figure 14. The discharge coefficients for both controlled and uncontrolled crests were computed and plotted in Figures 14 and 15. They were found to be satisfactory. The discharge coefficient for the controlled crest was compared with those of other controlled crests, Figure 16, and found to be about 3 percent higher than the average of the others.

INTRODUCTION

Boysen Dam is a part of the Boysen Unit of the Missouri River Basin Project. It is located in a canyon on the Big Horn River in central Wyoming, Figure 1. The dam, Figure 2, is an earth- and rock-fill structure approximately 1,100 feet long at the crest and has a height of about 150 feet above the bed of the river.

were made of wood and sanded smooth. The radial gates were made of 14-gage sheet metal and were pivoted on a single length of 1/8-inch round rod that extended from one training wall to the other through the center pier. The gate seal between the upstream face of the gate and the head wall, Figure 4, was a 1/8-inch-thick rubber flap attached to the model head wall. The surfaces of the training walls and the approach wing walls were made of sheet metal in the model.

The trashrack structure for the intake to the outlet tunnels was constructed of wood and covered with wire mesh to represent the trashracks. Tunnels, through which water was passed to the power plant and the outlet works stilling basin, were represented by two 2-1/2-inch metal pipes. In the model, one pipe supplied the power plant while the other supplied the outlet works as shown in Figure 6. A gate valve was installed in each for control of the discharges. The left wall of the powerhouse joined the right training wall of the spillway stilling basin, and the right wall of the powerhouse was the limit of the model in the transverse direction to the right. Openings from draft tubes and from the outlet works stilling pool were made to scale in the downstream face of the powerhouse. A separate model of the outlet works stilling basin was also constructed and tested, and is discussed in Hydraulic Laboratory Report No. Hyd-283*. The model powerhouse was partitioned into two sections—one section for passing the power plant discharge and the other for passing the outlet works discharge. Both the power plant and the outlet works sections were provided in the model with rock baffles to insure uniformly distributed flow leaving the power plant structure.

An erodible bed was included in the model to determine the relative extent and depth of erosion occurring for one stilling basin design as compared to that occurring for another. This bed, extending downstream from the end of the apron, shown in Figure 7, was molded in sand except for a rock outcrop shown in Section H-H of Figure 3. This exposed rock was believed capable of withstanding the velocities and erosion effects expected from the spillway operations, and consequently, it was molded of concrete mortar in the model as shown in Section C-C of Figure 5. The right and left banks of the river channel downstream from the stilling basin, shown in Figure 8, were also molded of concrete mortar placed on metal lath and shaped to the prototype river contours. A sample of the sand used in the erodible bed had the following analysis:

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*Hyd-283 "Hydraulic Model Studies of Boysen Outlets" by E. J. Rusho

A concrete spillway 66 feet wide, Figures 3 and 4, is located in the right abutment. The spillway crest is at elevation 4700, 52 feet below the maximum water surface of the reservoir, or 25 feet below the normal water surface. The flow is controlled by two 30- by 25-foot radial gates that are limited to a maximum vertical opening of 16 feet, as shown in Figure 4. The spillway is designed to pass a maximum discharge of 20,000 second feet which corresponds to a discharge of 333 second feet per foot of crest length, with heads ranging from 24 to 52 feet. Flow drops a vertical distance of 106 feet in a horizontal distance of 279 feet measured from the axis of the crest to the upstream end of the stilling basin floor. The stilling basin is 66 feet wide by 103 feet long, measured from the upstream end of the stilling basin floor to the downstream edge of the dentated end sill. Chute blocks located at the upstream end of the stilling basin are 4 feet high. The dentated end sill is 19 feet long in the direction of flow and the dentils on the sill are 8 feet 9 inches high. The floor of the stilling basin is a concrete horizontal apron that extends 48 feet beyond the end sill. Stilling basin training walls of the preliminary design rose 41 feet above the stilling basin floor; but as the result of model tests, a portion of the right training wall was raised 3-1/2 feet higher and turned inward 3 feet as shown in Section G-G of Figure 3.

The powerhouse, which houses the stilling basin of the outlet works as well as the power plant, is located immediately to the right of the spillway stilling basin as shown in Figures 2 and 3. The outlet tunnel intake and trashrack structure was located originally as shown in the model lay-out, Figure 5, but was relocated as shown in Figures 2 and 3.

THE MODEL

The model shown in Figures 5 through 9 was a 1:48 scale reproduction of the spillway and surrounding area. It was constructed and tested in the Bureau of Reclamation Hydraulic Laboratory at the Denver Federal Center. The reservoir was reproduced for a distance of 320 feet upstream from the spillway and the river channel for a distance of 430 feet downstream from the end of the apron.

Topography in the reservoir area of the model was molded of concrete mortar placed on metal lath. Model surfaces simulating non-concrete surfaces on the prototype, such as topography, were given a rough finish while surfaces simulating prototype concrete were finished smooth. The concrete spillway and stilling basin were molded in cement mortar using sheet metal templates to accurately define the surface. Piezometers of 1/16-inch-inside-diameter copper tubing, soldered to the center template of the right spillway bay, were placed flush and normal to the surface. The center pier, chute blocks, and the dentated sill

SIEVE ANALYSIS OF SAND SAMPLE

Passing a No. 4 sieve . . .	100 percent
Passing a No. 8 sieve . . .	91 percent
Passing a No. 16 sieve . . .	63 percent
Passing a No. 30 sieve . . .	27 percent
Passing a No. 50 sieve . . .	3 percent
Passing a No. 100 sieve . . .	0 percent

Water was supplied to the model by a portable 6-inch pump through an 8-inch line. The discharges were measured by an 8-inch orifice Venturi meter placed in the supply line. The reservoir and tail-water elevations were measured by a hook gage within a well and a point gage, respectively, and were located in the model as shown in Figure 5. The tail-water elevation was controlled by an adjustable gate at the extreme downstream end of the model. Model tail-water settings were determined from the tail-water curves shown in Figure 3.

THE INVESTIGATION

The investigation was a study of flow conditions throughout the structure which included: the study of flow conditions in the spillway approach including the effect of relocating the trashrack structure; the study of flow over the crest, including the calibration of the controlled crest for determining the gate opening necessary to maintain 20,000 second feet discharge; the study of flow in the stilling basin, including the testing of the preliminary design and six other designs which were modifications of the baffle pier arrangement in the preliminary basin; and the testing of two modifications of the stilling basin training wall design. The investigation was primarily concerned with the performance of the spillway discharging a maximum capacity of 35,000 second feet with the reservoir at maximum level, elevation 4752 and with the gates open to the 16-foot limit. Consideration was also given to the behavior of the spillway discharging 20,000 second feet since this was to be the normal maximum quantity of flow. After the first six stilling basin designs had been tested and just prior to the testing of the recommended design, it was learned that the flow was never to exceed 20,000 second feet, unless the spillway gates were unintentionally opened to more than that specified for maintaining 20,000 second feet or unless an emergency condition developed.

Spillway Approach

The spillway approach topography and the location of the trashrack structure at the intake portal to the outlet works and powerhouse tunnels as originally planned is shown in the model lay-out of Figure 5 and in the photograph of Figure 9(a). After tests had been made on the original arrangement, the intake portal and trashrack structure were relocated by the designers as a result of field investigations which disclosed that the foundation conditions in the original location were not as satisfactory as expected. The final location shown in Figure 6, and by photograph in Figure 9(b), proved more favorable even though the structure was more directly in the approach to the spillway.

Flows were observed for both locations of the trashrack structure. The latter location proved to cause less disturbance in the spillway approach flow as can be seen by comparing the water surface roughness in the right approach to the spillway in Figures 10(a) and 10(b). Apparently, when the trashrack structure was in its original location, flow over the underwater embankment between the spillway approach channel and the trashrack structure shown in Figure 9(a) caused the disturbance. When the trashrack was moved to its latter location, the embankment was removed, resulting in better flow conditions in the right approach to the spillway. Free flow over the crest was measured with the trashrack structure in each location to determine whether the spillway discharge had been affected by the change. The measurements showed that after relocation of the trashracks the discharge of the spillway was increased slightly with heads of 9 feet or more as shown in Figure 11.

When calibrating the spillway crest with the trashrack structure in its latter location and the gates fully open, it was found that with headwaters above elevation 4710 the reservoir surface came in contact with the bottom edge of the left radial gate before that of the right, indicating that the trashrack structure was directing a part of the flow into the left gate opening. This observation was confirmed by photographing the flow lines in this area as shown in Figure 12. The flow lines were photographed using a 1/2-second exposure. They show clearly that the trashrack is directing flow, which would normally enter the right bay, into the left bay; thereby, causing the water at the left bay to pile up, so to speak, to a slightly greater elevation. However, since the difference in quantity of discharge in the two bays was small and caused no unfavorable flow problems of any consequence in any part of the structure, no further efforts were made to improve the entrance conditions. Furthermore, since the flow normally is controlled, the pooling effect caused by the partially open gates reduces the unequal flow distribution to a negligible amount. The new location of the trashrack structure was therefore considered satisfactory from a hydraulic standpoint.

Spillway Crest and Gates

Pressures on the spillway crest were recorded for flows of 20,000 second feet with reservoir elevations 4724, 4734, 4743, and 4752 and the corresponding necessary gate openings. All pressures recorded were above atmospheric, except just downstream from the crest in the vicinity of Station 6+06.3, where the pressure dropped to 0.7 foot of water below atmospheric when the reservoir was at maximum elevation 4752, as shown in Figure 13.

With the trashrack relocated, spillway discharge curves for the uncontrolled crest and for a gate controlled crest with openings of 6, 8, 10, 12, 14, and 16 feet were obtained from model calibration tests and are shown in Figure 14. The maximum possible discharge was found to be 34,800 second feet when the gates were open the maximum limit of 16 feet and the reservoir was at maximum elevation 4752. However, it is planned that the prototype spillway discharge will be controlled by setting the gates to maintain a discharge of not more than 20,000 second feet at all times. Both gates are to be opened the same amount. The gate opening required to maintain 20,000 second feet for any reservoir elevation between 4724 and 4752 was determined and is shown by the gate opening curve for Q = 20,000 second feet in Figure 14.

Using the gate controlled discharge curves in Figure 14, the value of the discharge coefficient was computed for the controlled crest in the equation: $Q = 2/3\ C_d L 2g\ (H^{3/2} - h^{3/2})$, where Q is the total discharge in cubic feet per second, C_d is the discharge coefficient, L is the crest length excluding the width of pier, H is the difference in elevation between the reservoir water surface and the crest, and h is (H-D) where D is the gate opening; the gate opening being the difference in elevation between the spillway crest and the lower edge of the gate. The coefficient, C_d, was plotted against H/D, as shown in Figure 15. Figure 7 of Hydraulic Laboratory Report No. Hyd-109*, reproduced in this report as Figure 16, shows similar coefficients obtained for six other gate-controlled crests. A comparison of Figures 15 and 16 shows that the controlled crest coefficients for Boysen Dam spillway are approximately 3 percent higher than the average of those for the others.

The discharge coefficient was also computed for an uncontrolled crest using the equation: $Q = CLH^{3/2}$, where C is the discharge coefficient. The coefficient plotted against head is shown in Figure 14. At about reservoir elevation 4720, where the gates begin to control the flow, the discharge coefficient of the free crest is 3.30, which under other circumstances might be considered low. Since the flow normally is to be controlled by the gates, it was not practical to attempt to increase the coefficient of discharge for the uncontrolled crest.

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*Hyd-109 "Hydraulic Model Studies of Granby Dam Spillway," Colorado-Big Thompson Project, by R. R. Pomeroy

Since the discharge could be controlled as desired and since no appreciable subatmospheric pressures were observed on the crest, the preliminary spillway crest and gate arrangement shown in Figure 4 was considered satisfactory.

Preliminary Stilling Basin

Description. The preliminary stilling basin, shown in Figure 3 and in plan in Figure 17, was 66 feet wide by 103 feet long from chute blocks to downstream edge of the dentated end sill plus 48 feet of concrete apron extending beyond the end sill. Training walls along each side of the stilling basin rose 41 feet from the basin floor. The downstream end of the right-hand training wall of the stilling basin joined the upstream end of the left powerhouse wall.

Normally, the maximum quantity of flow was to be 20,000 second feet, while the maximum capacity of the spillway was 35,000 second feet discharging through two 16-foot gate openings with a head on the crest of 52 feet. The maximum flow corresponds to a unit flow entering the stilling basin of 530 second feet per foot of width. Length of the stilling basin from chute blocks to downstream edge of dentated end sill in terms of 35,000 second feet was $2.5d_2$, where d_2 is the difference in elevation between the tail water and stilling basin floor. Length of the stilling basin from chute blocks to upstream face of dentils was approximately $2d_2$ and the height of dentils on the dentated end sill was approximately $1/5d_2$. Therefore, for the maximum flow of 35,000 second feet, it is apparent that the stilling basin is small, but because of foundation and other conditions it was found by the designers to be very uneconomical to design it longer, wider, or deeper.

For 20,000 second feet the tailwater elevation is less, thus d_2 is smaller; therefore, the length of stilling basin from chute blocks to downstream edge of dentated end sill was $3d_2$, while the height of the dentils on the dentated sill was $1/4d_2$. The flow entering the stilling basin was about 300 second feet per unit foot of width. For the normal flow it is thus apparent that the basin proportions are more nearly correct.

General performance. The stilling basin was tested for flows of 20,000 and 35,000 second feet. The water-surface through the stilling basin and immediately downstream was rough for discharges of 20,000 second feet and exceedingly rough for 35,000 second feet, as shown in Figures 18 and 19. For 35,000 second feet a very high boil can be observed over the dentated sill. The average water-surface profile measured along the right training wall is shown in Figure 20 for both 20,000 and 35,000 second feet.

Right training wall. The right-hand training wall joining the left-hand powerhouse wall was overtopped by the splash and boil in the stilling basin for discharges of 20,000 second feet and greater. One purpose of this training wall was to protect an area behind the powerhouse which was backfilled with rock and drained by a 4-inch drain to the powerhouse sump. Therefore, it was desirable to keep the quantity of flow overtopping the right-hand training wall to a minimum. To remedy the overtopping condition, the wall extending upstream from the powerhouse was increased 16 feet in height for a distance of 40 feet. This proved to be ample for the maximum possible discharge of 35,000 second feet and was adopted for use with the next five succeeding stilling basin designs tested.

The wall recommended for prototype construction, however, was developed after completion of tests on five of the succeeding stilling basins, for it was not learned until then that the stilling basin would seldom if ever be required to handle more than 20,000 second feet; and, therefore, a more economical wall could be used. The preliminary wall, increased 3-1/2 feet in height, was combined with a sea wall lip projecting inward 3 feet, as shown in Section G-G of Figure 3, and extended 48 feet upstream from the powerhouse. This wall, recommended for the prototype, confined the waves to the stilling basin and was ample for discharges up to 20,000 second feet. If, inadvertently, higher discharges occurred the wall would also aid in confining the boil to the stilling basin. Unless the preliminary wall was increased at least 3-1/2 feet in height the projecting sea wall lip proved to be relatively ineffective. The recommended wall thus served the purpose of the higher wall and was more economical to construct.

Hydraulic jump stability. Stability of the hydraulic jump in the basin of the preliminary design was considered adequate. For a flow of 35,000 second feet, it was possible to lower the tail water 6 feet below the normal expected tail water before the hydraulic jump was swept from the apron. Once the jump was swept from the apron, it was necessary to raise the tail water to 3 feet below the normal expected tail water before the jump fell back on the apron.

Erosion. The purpose of the scour tests was (1) to evaluate the erosion forces downstream from the end of the apron existing in one design of the stilling basin and compare them to those existing in another, and (2) to determine whether eroded material sufficient to affect powerhouse operation would deposit at the draft tube exits as a result of spillway operation. Thirty-minute model scour tests were run, each beginning with the original riverbed contours formed in sand as shown in Figure 7. A rock ledge along the powerhouse wall and extending downstream from the end of the apron, as shown in Figure 5, was formed of concrete in the model since it was believed that the rock ledge would resist erosion in the prototype.

The first scour test was run with a discharge of 20,000 second feet, Figure 21(a). The maximum depth of scour was to elevation 4590 near the end of the apron or 4 feet below the apron elevation. A small amount of sand was carried upstream and deposited on the apron near the powerhouse side of the stilling basin. No sediment was deposited near the draft tube exits, instead some scour occurred.

A second scour test was run with a discharge of 35,000 second feet, Figure 21(b). The maximum depth of scour was to elevation 4587 or 7 feet below the apron elevation over a larger area downstream and to the left of the apron. The scour downstream to the left of the apron was along the toe of the dam and was caused by a large but slow velocity eddy in that area. The erosion was considered excessive. No material was deposited near the draft tube exits; instead, scour occured to a depth of 1 foot.

Stilling basin erosion tests were also run with the power plant discharging its maximum flow of 2,300 second feet and the outlet works discharging its maximum of 1,480 second feet while the spillway was discharging 20,000 second feet for one test and 35,000 second feet for another. In both tests, the scour pattern near the draft tubes was little affected by the power plant and outlet works discharges. Succeeding scour tests with various stilling basin designs also showed no deposition of material near the powerhouse draft tubes.

Modification Limitations. Since at this stage of the investigation it was required that the stilling basin be capable of handling 35,000 second feet of flow, it was considered necessary to modify the stilling basin design in order to elimate the excessive water-surface roughness, the high boil over the dentated sill, and the large eddy causing erosion downstream and to the left of the stilling basin. This could be accomplished by either lengthening, widening, or perhaps deepening the basin; but, it had been pointed out by the designers that to do any one of these three would be exceptionally costly for prototype construction. Therefore, in the succeeding six stilling basin designs, attempts to improve the performance of the stilling basin were made by rearrangement of the sill and baffles.

Stilling Basin Design 2

Design 2, shown in Figure 17, used a smaller dentated sill. The sill was reduced to one-half the size of the preliminary one in order to reduce the height of the boil over the dentated sill and to smooth out the surface roughness.

Flow conditions for a discharge of 35,000 second feet are shown in Figure 22(a). The water surface was not nearly as rough as for the preliminary design, Figure 19. Water-surface profiles along the right training wall for discharges of 20,000 and 35,000 second feet were not nearly as high throughout the basin as those of the preliminary design as shown in Figure 20. The stability of the jump, however, was not good for flows of 35,000 second feet, as the jump was swept from the apron when the tail water was lowered only 0.2 of a foot below normal expected tail water and it was necessary to raise the tail water 0.6 of a foot above normal tail water before the jump fell back on the apron. An erosion test with a discharge of 20,000 second feet showed that scour was slightly less severe than that which occurred for the preliminary design, as can be seen by comparing Figure 22(b) with Figure 21(a). For 35,000 second feet considerable scour was caused, as in the preliminary design, by the large eddy downstream and to the left of the stilling basin which appeared to be slightly swifter than the eddy observed for the preliminary design.

Stilling Basin Design 3

Design 3, shown in Figure 17, used a modification of Design 2. The half-size dentated sill of Design 2 was moved upstream 20 feet in order to move the jump upstream for fuller use of the upper end of the basin; and therefore, reduce the maximum height of the water surface profile.

Flow conditions with a discharge of 35,000 second feet are shown in Figure 23(a). The water surface was very rough, much rougher than for the preliminary design, and it was impossible to get an accurate measurement of the water-surface profile. With 20,000 second feet, the water surface was somewhat smoother than for the preliminary design and the boil was not quite as high as can be seen by the water-surface profiles in Figure 20. Again the jump was found to be unstable for a flow of 35,000 second feet. It was swept from the apron when the tail water was lowered only 0.2 of a foot below the normal expected tail water. It was necessary to raise the tail water 0.7 of a foot above normal tail water in order to bring the jump back into the basin. An erosion test was run for a flow of 20,000 second feet. Scour was 4 feet deeper and covered a larger area than that occurring for the preliminary design as can be seen by comparing Figure 23(b) with Figure 21(a). For 35,000 second feet scour was caused by an eddy similar to that of the preliminary design.

Stilling Basin Design 4

Design 4, shown in Figure 17, used a modification of the preliminary design. A dentated sill was used, which was the same height and width as the preliminary, but contained a greater number of narrower dentils. Narrower dentils provided about 25 percent more space between dentils through which the flow could pass. The purpose of the modification was to lower the height of boil over the dentated sill and to reduce surface roughness. The location of the dentated sill and other features of the preliminary stilling basin were unaltered.

Flow conditions for 35,000 second feet are shown in Figure 24(a). The water surface downstream from the basin was smoother than that of the preliminary as can be seen by comparing Figure 24(a) with Figure 19. Water-surface profiles along the right training wall were considerably lower than those of the preliminary design for discharges of both 20,000 and 35,000 second feet as can be seen in Figure 20. Sweep-out tests, however, proved the jump to be not so stable as in the preliminary. For a flow of 35,000 second feet, the jump was swept from the apron when the tail water was lowered 3.2 feet below normal expected tail water, and the jump did not fall back on the apron until the tail water was raised to 1.5 feet below normal. An erosion test was run with a flow of 20,000 second feet. Scour was 2 feet deeper and covered a larger area than that which occurred for the preliminary design as can be seen by comparing Figure 24(b) with Figure 21(a). For 35,000 second feet the eddy downstream and to the left of the stilling basin was a little swifter than that observed in the preliminary design.

Stilling Basin Design 5

Design 5, shown in Figure 17, used the preliminary design with an additional intermediate dentated sill located between the chute blocks and the existing dentated sill. The purpose of the modification was to fill the upstream portion of the stilling basin more completely and thereby reduce the boil height downstream. The intermediate sill was one-half the size of the end sill.

Flow conditions showed the hydraulic jump to be less stable than for any design tested. For 35,000 second feet the jump was swept from the apron even at normal tail water. The jump left the apron at the intermediate dentated sill as shown in Figure 25(a). For a discharge of 20,000 second feet, the jump filled the basin more completely than for any design thus far tested. A water-surface profile for 20,000 second feet was recorded and plotted in Figure 20. It shows that the stilling basin is being put to fuller use. An erosion test for a discharge of 20,000 second feet showed that scour was 5 feet deeper and covered a larger area than that which occurred for the preliminary design as can be seen by comparing Figure 25(b) with Figure 21(a).

Stilling Basin Design 6

For Design 6, shown in Figure 17, the intermediate sill of Design 5 was replaced with baffle piers that were of the same size and spacing as the dentils on the intermediate sill. This was done because it was apparent that the sill of Design 5 was deflecting the high-velocity on the floor of the stilling basin upward into the air, causing the jump to sweep out.

Flow conditions for discharges of 20,000 and 35,000 second feet are shown in Figures 26 and 27, respectively. For both discharges, the water surface was smoother than that in the preliminary design as can be seen by comparing Figures 26 and 27 with Figures 18 and 19. For both discharges the water-surface profile was somewhat lower than for the preliminary design. For 35,000 second feet, stability of the jump was not good, as the jump was swept from the apron when the tail water was lowered only 3.3 feet below normal expected tail water, and it was necessary to raise the tail water 2.2 feet above normal in order to bring the jump back on the apron. Erosion tests for 20,000 and 35,000 second feet were run. Scour for 20,000 second feet was 1 foot deeper and covered a slightly larger area than that of the preliminary design as can be seen by comparing Figure 28(a) with Figure 21(a). Depth of scour for 35,000 second feet was 2 feet less than that of the preliminary design and not so extensive as can be seen by comparing Figure 28(b) with Figure 21(b). The large eddy downstream and to the left of the stilling basin was hardly distinguishable; and therefore, very little erosion occurred in that area.

Recommended Stilling Basin

At this stage in the investigation it was learned that the stilling basin would not be required to handle more than 20,000 second feet unless the spillway gates were accidentally opened greater than specified by the gate opening curve of Figure 14 or unless an emergency developed. Therefore, and because of an immediate need for field drawings, the designers adopted the preliminary stilling basin design for prototype construction. The sill and baffle pier arrangement of that design was considered satisfactory for flows up to 20,000 second feet and was better than any arrangement thus far tested. However, testing was continued in an effort to improve the basin performance for the expected maximum flow of 20,000 second feet and for emergency operation at 35,000 second feet. As a result, the recommended design was developed but not in time for use on the prototype.

Description. In the design recommended by the laboratory, the dentated sill of the preliminary design was relocated 24 feet downstream from its preliminary position as shown in Figure 17, making the total length of the stilling basin from chute blocks to downstream edge of end sill 144 feet, or, for 35,000 second feet about 3.5 (d_2) as compared to 2.5 (d_2) in the preliminary design. This was the limit to which the sill could be moved downstream without lengthening the left training wall. Lengthening this wall was considered economically impractical by the designers. It was expected that moving the sill downstream would also move the high point of the boil downstream so that it would occur adjacent to the high powerhouse wall rather than farther upstream adjacent to the lower training wall.

General performance. Flow conditions for discharges of 20,000 and 35,000 second feet are shown in Figures 29 and 30, respectively. Water surfaces for both discharges were much smoother than for the same discharges in the preliminary design as can be seen by comparing Figures 29 and 30 with Figures 18 and 19. However, for 35,000 second feet, the water surface was still rough indicating incomplete energy dissipation. It was, therefore, recommended that this basin be used to handle 35,000 second feet, only for emergency conditions. The height of boil for both 20,000 and 35,000 second feet was nearly the same as in the preliminary design but the high point of the boil was moved downstream so that it occurred adjacent to the powerhouse as shown in Figure 20. Consequently, with this location of the end sill the training walls were satisfactory as preliminarily designed.

Hydraulic jump stability. The stability of the jump was tested and found to be satisfactory. With a discharge of 35,000 second feet the tail water was lowered 6.8 feet below normal expected tail water elevation before the jump was swept from the apron and it was necessary to raise the tail water only 1.8 feet, or to 5 feet below normal before the jump fell back on the apron. For 20,000 second feet it was difficult to sweep the jump out of the stilling basin. These figures indicate a greater margin of safety against sweep out than for any other design tested.

Erosion. Scour test results with discharges of 20,000 and 35,000 second feet were almost identical to those of the preliminary design as can be seen by comparing Figures 31(a) and 31(b) with Figures 21(a) and 21(b), with the exception that for each test much more bed material was carried upstream and deposited on the apron near the powerhouse wall. This was probably due to the ground roller occurring at the end of the paving which might tend to prevent undermining of the apron. For flows of 35,000 second feet, excessive scouring occurred downstream and to the left of the stilling basin as was the case in most of the other designs. This erosion was again due to the large eddy in this area. For flows of 20,000 second feet the eddy and the scour resulting from it were nonexistent. No sediment was despoited at the exits of the draft tubes in either test.

Conclusion. Comparing the performance photographs of the various stilling basin designs, the water-surface profiles in Figure 20, and the data summarized in Figure 32; it is concluded that the recommended stilling basin provides the best protection to the powerhouse, the stilling basin itself, and to the river channel downstream. Therefore, it was recommended by the Hydraulic Laboratory for use in the prototype structure. However, the designers, pressed for field construction drawings, had previously adopted the preliminary design and the modified right training wall which was termed a satisfactory scheme for the anticipated maximum flow of 20,000 second feet. The designers felt that the design recommended by the laboratory did not show sufficient improvement in operation to warrant revision of construction which was already in progress.

Scheme One

INTRODUCTION

Prior to the investigation described in the preceding section of this report, a complete model study of the spillway initially proposed for Boysen Dam was made during the period January 1946 to November 1946. The structure proposed at that time was quite different from the more recent design described in the preceding section.

In Scheme One, Figure 33, the capacity of the spillway was to be 82,000 second feet, or approximately four times greater than that of the later scheme. The flow was to be discharged over a crest controlled by two 40- by 38-foot slide gates with a head of 65 feet; this amounts to a concentration of 1,030 second feet per linear foot of crest length. The spillway crest was connected to the stilling basin by a long, diverging spillway chute making the total length of the structure 600 feet from spillway crest to downstream edge of end sill as compared to 382 feet in the later scheme. The vertical drop from crest to stilling basin floor was 81 feet, compared to 206 feet in the second scheme. The stilling basin of Scheme One was 160 feet wide by 140 feet long, while the stilling basin of the Scheme Two was 66 feet wide by 103 feet long. Therefore, the average discharge per foot of width entering the stilling basin was about 560 second feet for the earlier scheme as compared to about 303 for the later one. Length of stilling basin in terms of d_2 in both schemes was approximately $3d_2$. The powerhouse was located on the left hand side of the spillway stilling basin. Other minor differences between Schemes One and Two may be seen by comparing Figures 33 and 3.

THE INVESTIGATION

Since Scheme One was abandoned in favor of Scheme Two, only that part of the investigation of the first scheme which might be of general use and interest will be discussed here. The model studies were made on a 1:60 scale model of the spillway.

Initial Spillway Design

Before the model of the initial design had been constructed the length of the chute was reduced 73 feet which reduced the over-all length of the structure from 600 feet, as defined above and shown in Figure 33, to 527 feet. Other features of the structure remained unaltered from those shown in Figure 33. The arrangement of the model used to test the initial design is shown in Figure 34, and photographs of the model are shown in Figure 35. The model in operation showed the flow in the spillway chute to be unequally distributed across its width which was particularly noticeable for the lower discharges. Flow entering the stilling basin was concentrated in the center causing eddies to form along each side of the basin. Since the basin was not being utilized properly or to its fullest extent, unfavorable scour patterns occurred in the channel downstream from the apron. To obtain better distribution of flow entering the basin and thereby eliminate the excessive eddy action, the training walls of the spillway chute were realined so that the chute divergence started at the spillway crest and continued uniformly to the point of curvature of the steep incline. At this point the training walls were turned parallel to the structure centerline and continued in this manner to and through the stilling basin, as shown in Figure 36(a). By this method the flow was expanded to the same width as before. Maximum width occurred upstream from the stilling basin, however, so that by the time the flow reached the stilling basin it was quite uniformly distributed across its width. Flow conditions in the model showed a definite improvement.

The spillway was calibrated for free flow over the crest and the coefficient of discharge computed using the formula $Q = CLH^{3/2}$. The crest was found to be satisfactory from the standpoint of spillway capacity. The spillway capacity and coefficient of discharge curves are shown in Figure 37. At this point in the investigation additional data were received from the field which indicated that the capacity of the spillway was too large. From the field data the maximum necessary capacity of the spillway was determined to be 62,000 second feet, compared to 82,000 second feet initially.

Second Spillway Design

Description. As a result of the reduced maximum capacity required of the spillway, the designers reduced its size. The width of the spillway at the crest and at the stilling basin was reduced to 70 feet and 125 feet, respectively. The center pier remained 10 feet wide so that the effective length of the crest was 60 feet. The discharge per foot of crest length was then approximately 1,030 second feet while the discharge entering the stilling basin per foot of width was about 500 second feet. The training wall modification that proved satisfactory in the initial spillway design was not incorporated in this design by the designers as it required more concrete and therefore was more costly. It was hoped that unequal distribution of flow would not be a problem in the stilling basin of the second design. Slide gates were replaced by radial gates and all length dimensions of the structure in the direction of flow remained the same as for Design One. The model lay-out of the second design is shown in Figure 38.

Performance. Tests proved the training wall modification described in the initial Spillway Design to be necessary for this design also. Therefore, the model was modified with the divergence starting at the spillway crest and ending at the point of curvature of the incline, as shown in Figure 36(b).

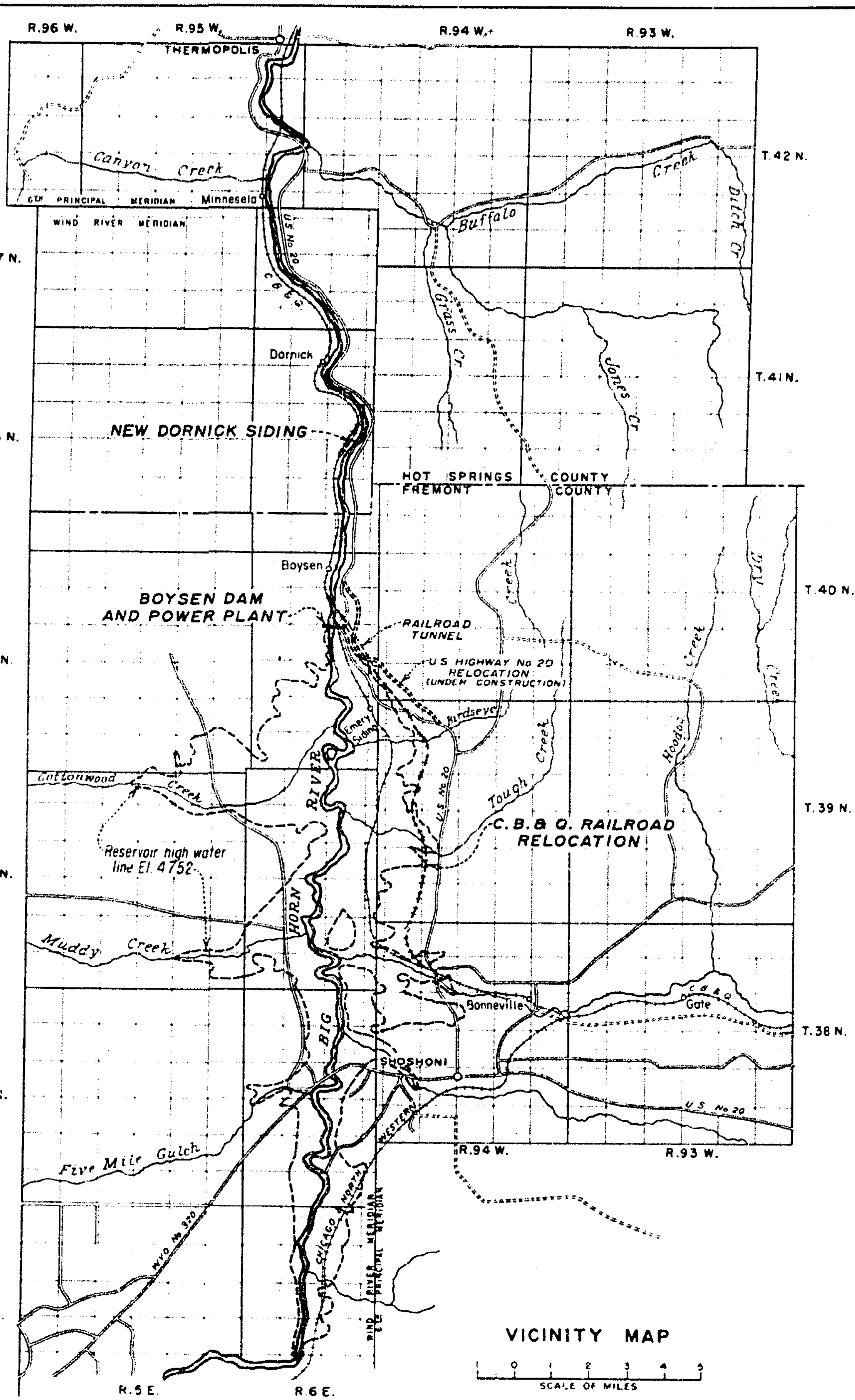
A fin which formed at the downstream end of the center spillway pier was almost entirely eliminated by reducing the thickness of the pier from 10 to 6 feet. Reducing the pier 4 feet in thickness made possible the reduction in width of the spillway at the gate section by the same amount. This reduction of pier thickness was possible since gate slots in the pier were eliminated by the switch from slide to radial gates. The narrower pier, in addition, caused the high point of the water-surface profile along the training walls of the chute to be reduced, making it possible, therefore, to reduce the height of the chute training walls.

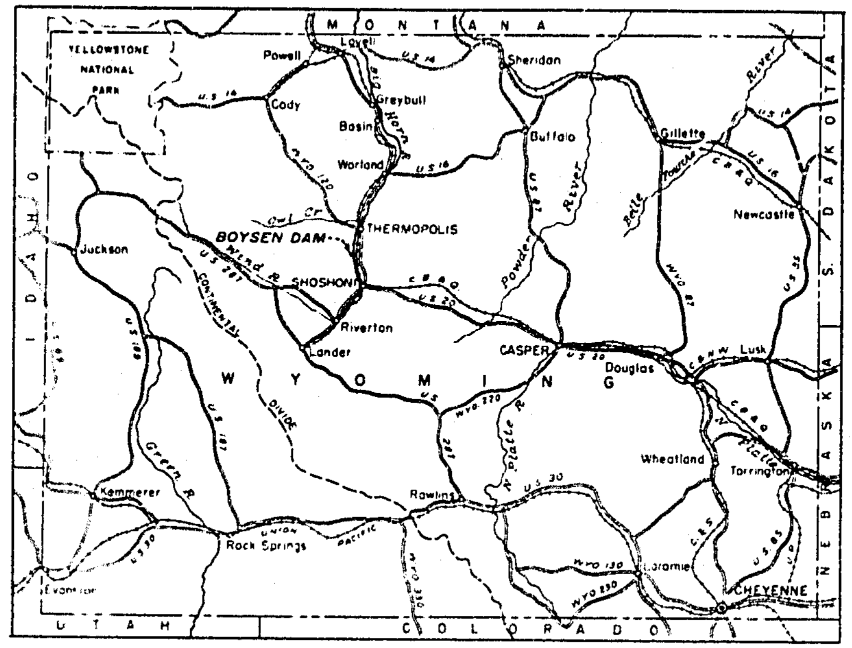
Second design modified. The second design of the spillway with training wall and gate pier modifications as described in the preceding paragraph incorporated into the design is shown in Figure 39. Photographs of the model with these two modifications installed are shown in Figure 40.

With these modifications installed in the model, the spillway was calibrated for free discharge over the crest and the coefficient of discharge computed using the formula $Q = CLH^{3/2}$. The spillway capacity curve obtained from the calibration and the coefficient of discharge curve are shown in Figure 37. The spillway capacity curve shows a reduction in discharge over that of the initial design which is due principally if not entirely to the shorter length of crest since the discharge per unit foot of crest length is nearly the same in both cases. The coefficients of the two designs differ by less than 2 percent and for practical purposes may be considered identical. It is to be expected that the coefficients of both should be identical or nearly so since the crest shape of both designs are identical.

It was necessary that a wall be provided along the right side of the channel downstream from the stilling basin in order to give adequate protection to the railroad right of way. The height and length of this wall which extended beyond the stilling basin along the right embankment was determined by the model study. The minimum size of riprap to protect the riverbed and for placement on the side slopes just downstream from the stilling basin was also determined by model test. Information which might eventually be of use in establishing general design procedures includes a water-surface profile along the centerline of the chute shown in Figure 41 for the maximum discharge with the gates fully open, and cross sectional water-surface profiles, shown in Figure 42, taken at 60-foot intervals throughout the steeper portion of the spillway chute and the stilling basin.

FIGURE 1





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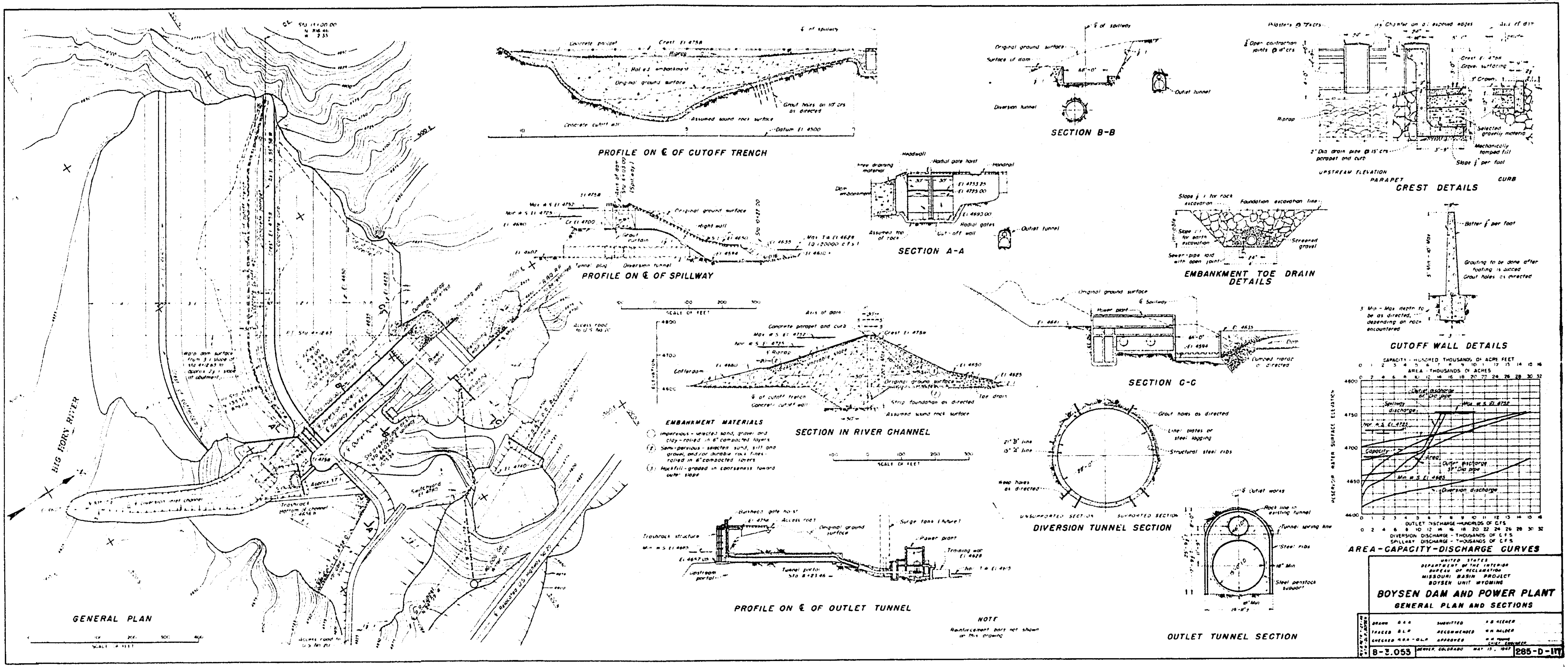
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MISSOURI BASIN PROJECT
BOYSEN UNIT-WYOMING

BOYSEN DAM AND POWER PLANT

LOCATION MAP

DRAWN D.A.A.	SUBMITTED T.W. Mermel
TRACED C.M.A.	RECOMMENDED W.H. Nalder
CHECKED H.G.A. Md.	APPROVED Walker R. Young, CHIEF ENGINEER
DENVER, COLORADO JULY 2, 1946	285-D-110

FIGURE 2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AREA-CAPACITY-DISCHARGE CURVES
NOTE
Reinforcement bars not shown on this drawing
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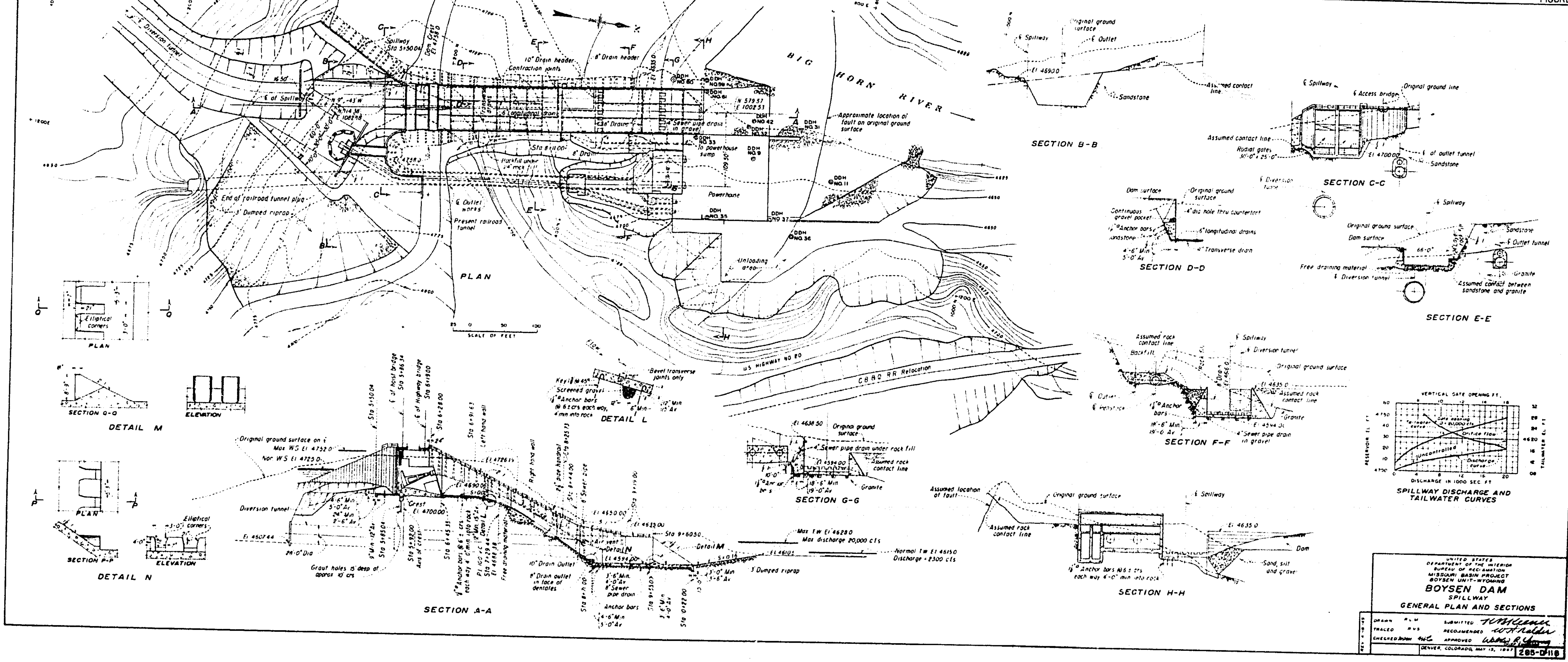
285-D-117

FIGURE 3



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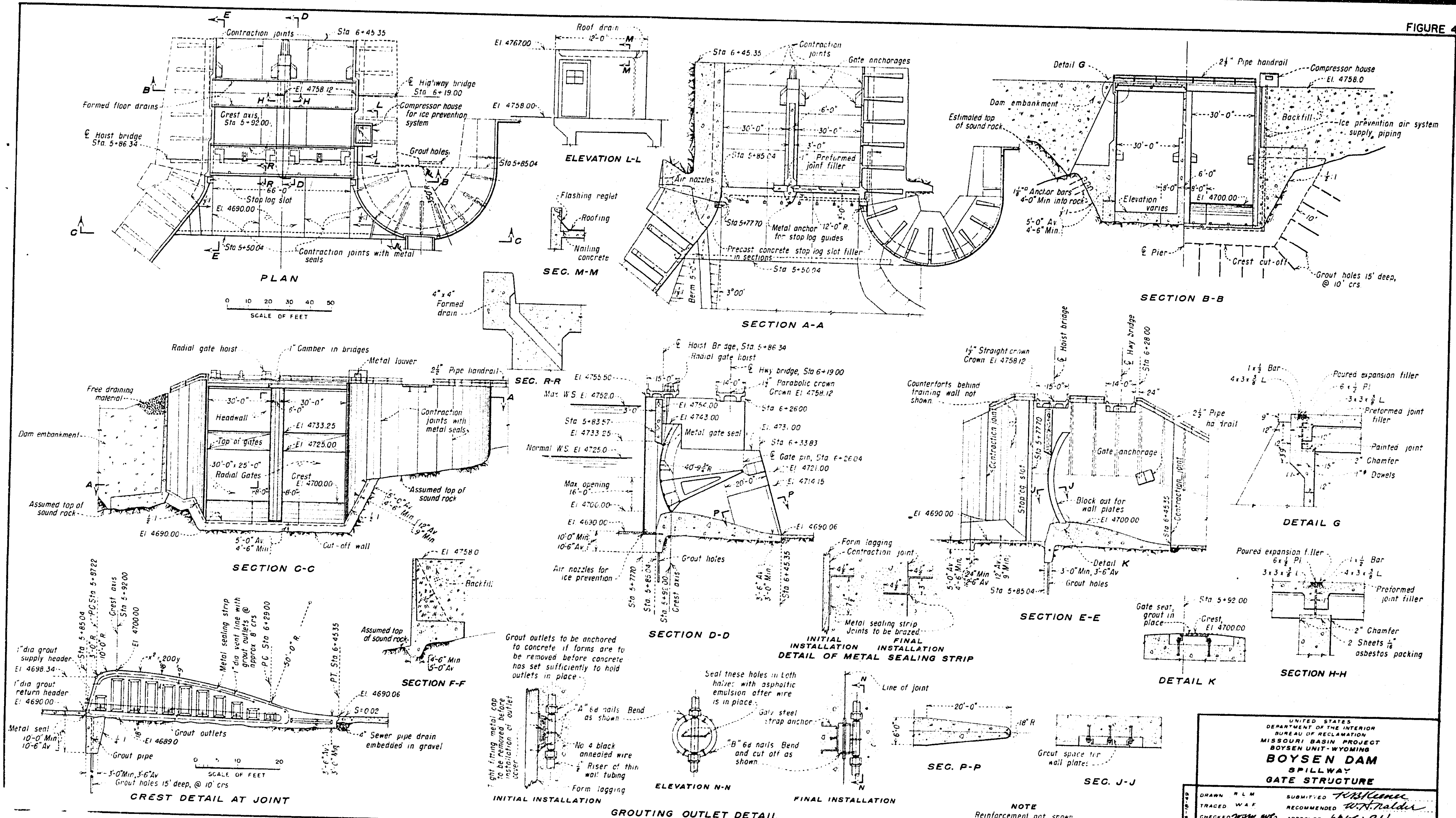
FIGURE 4
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SECTION B-B
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SEC. J-J
NOTE
Reinforcement not shown
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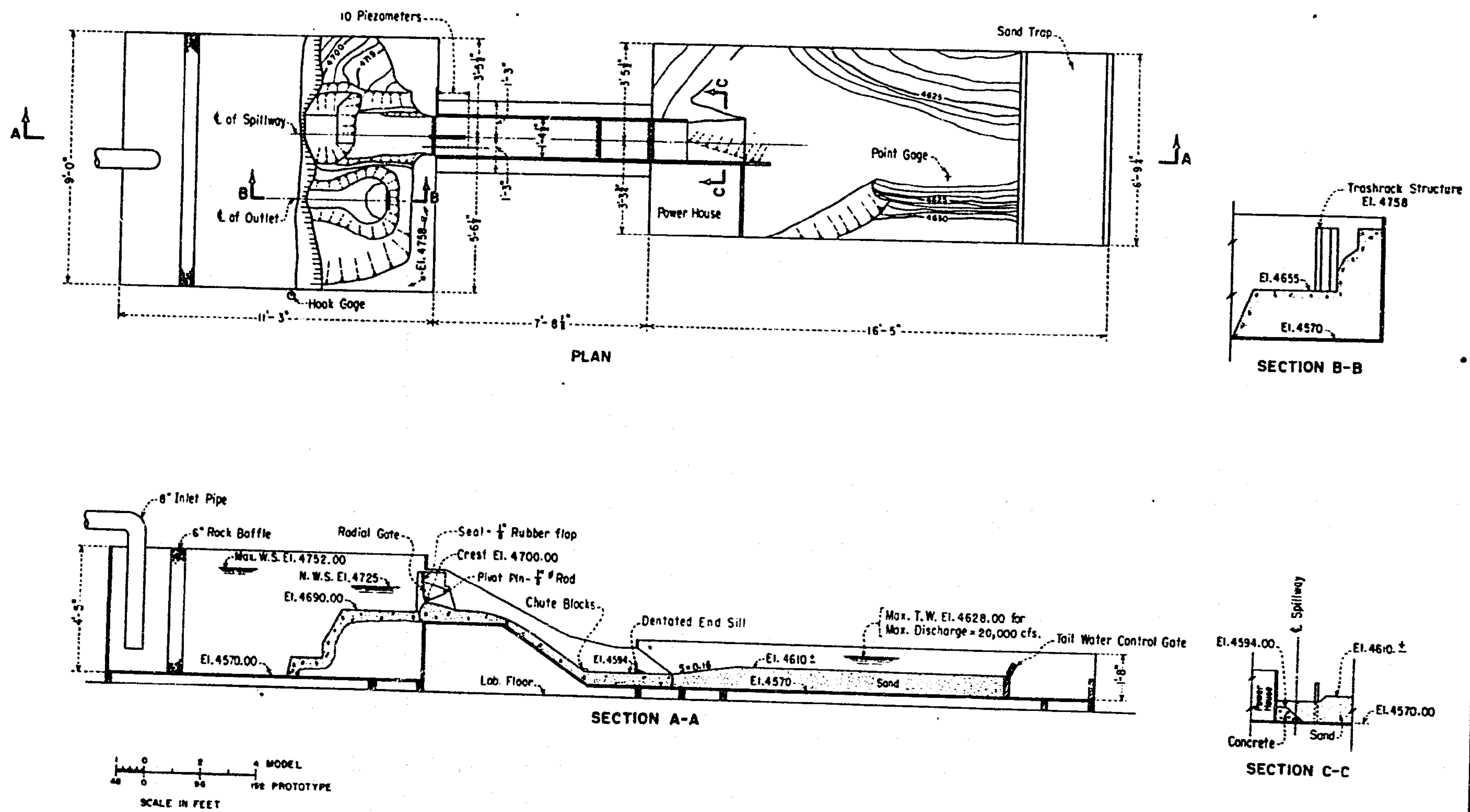
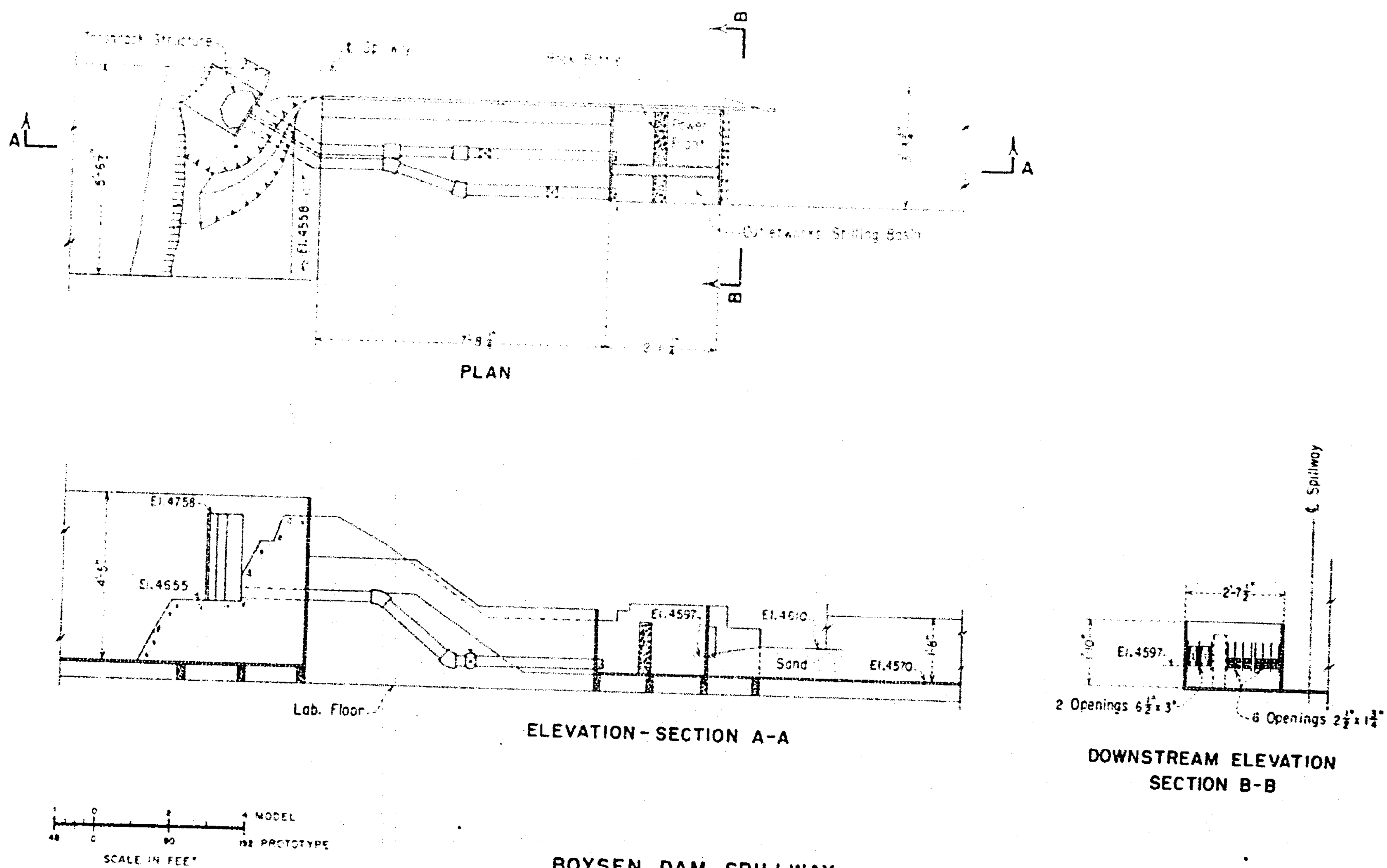
BOYSEN UNIT-WYOMING
BOYSEN DAM
SPILLWAY
GATE STRUCTURE
DRAWN R.L.M.
TRACED W.A.F.
CHECKED
SUBMITTED
RECOMMENDED
APPROVED
CHIEF ENGINEER
DENVER, COLORADO, MAY 15, 1947
285-D-119



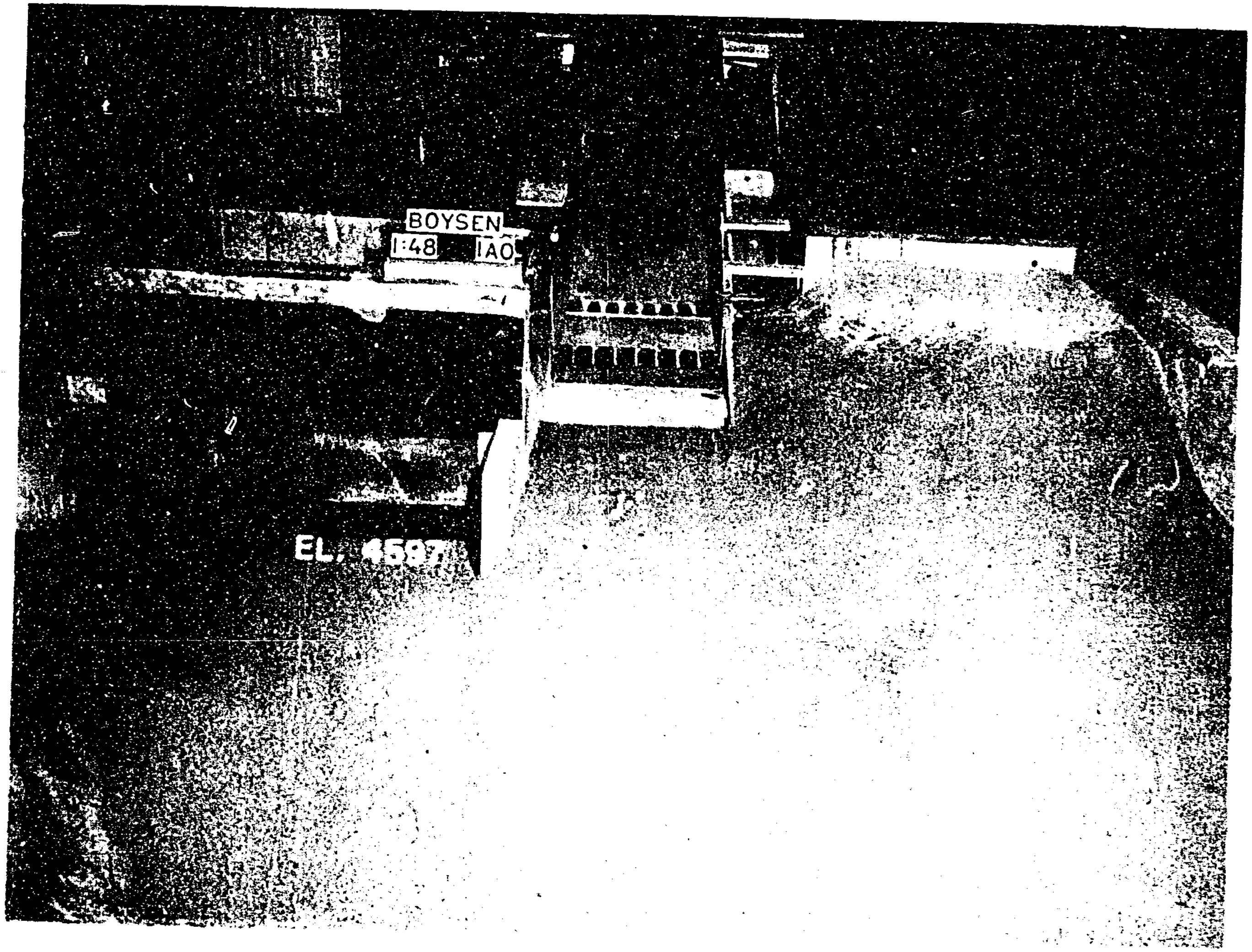
BOYSEN DAM SPILLWAY
MODEL LAYOUT
1:48 MODEL



BOYSEN DAM SPILLWAY
MODEL LAYOUT REVISION
1:48 MODEL

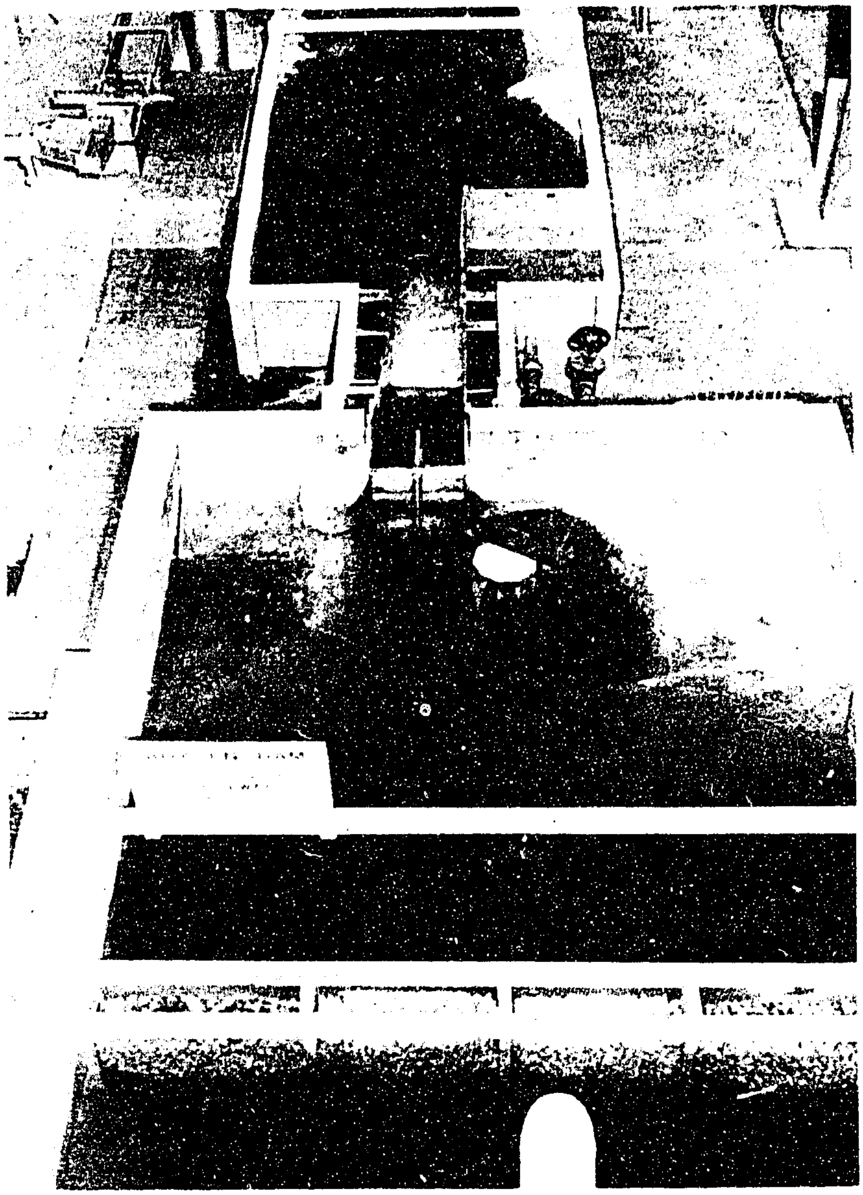
FIGURE 6

Figure 7



BOYSEN DAM SPILLWAY
Model Stilling Basin Area
1:48 Model

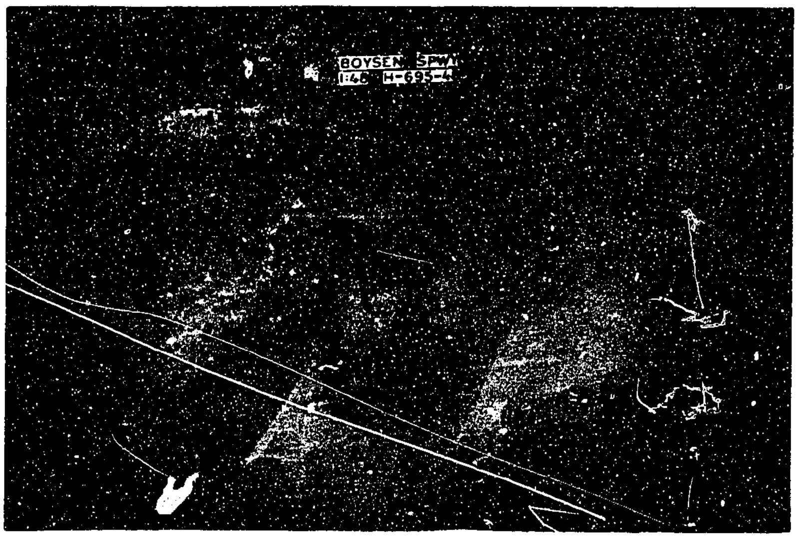
Figure 8



Discharge - 20,000 Second-feet

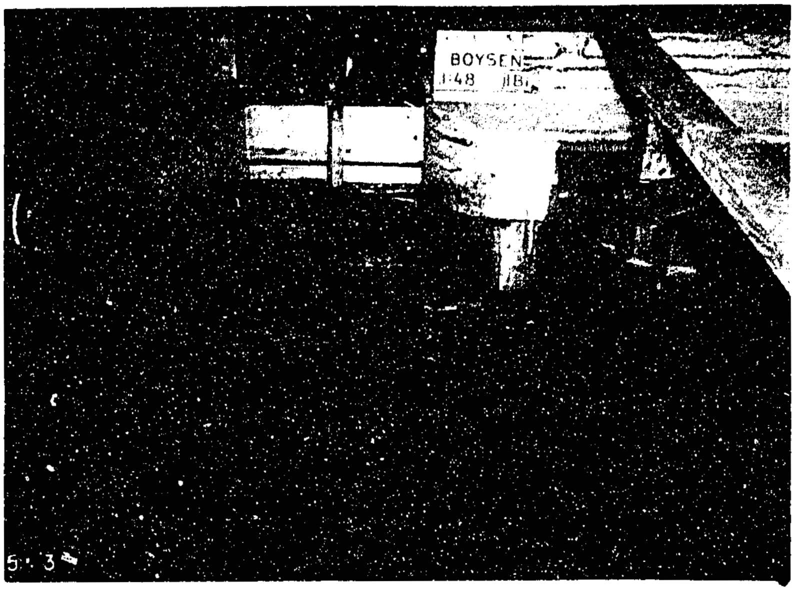
BOYSEN DAM SPILLWAY
Model In Operation
1:48 Model

(a) Original trashrack location and topography in spillway approach.

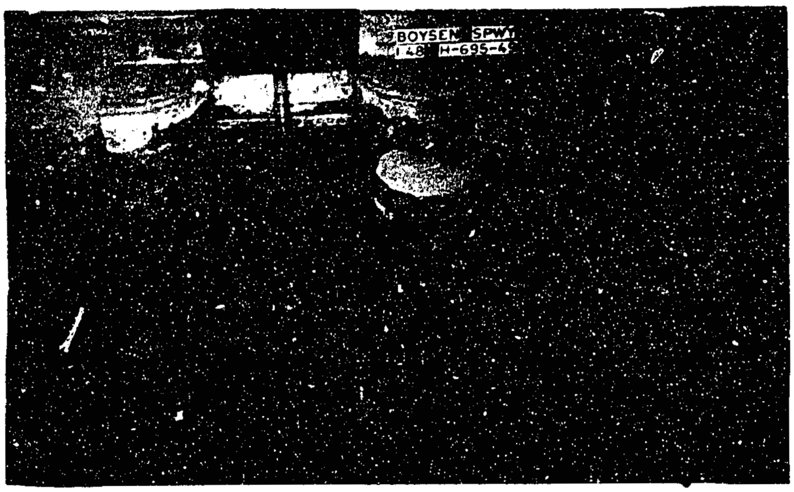


(b) Final trashrack location and topography in spillway approach.

BOYSEN DAM SPILLWAY
Trashrack and Topography in Spillway Approach
1:48 Model

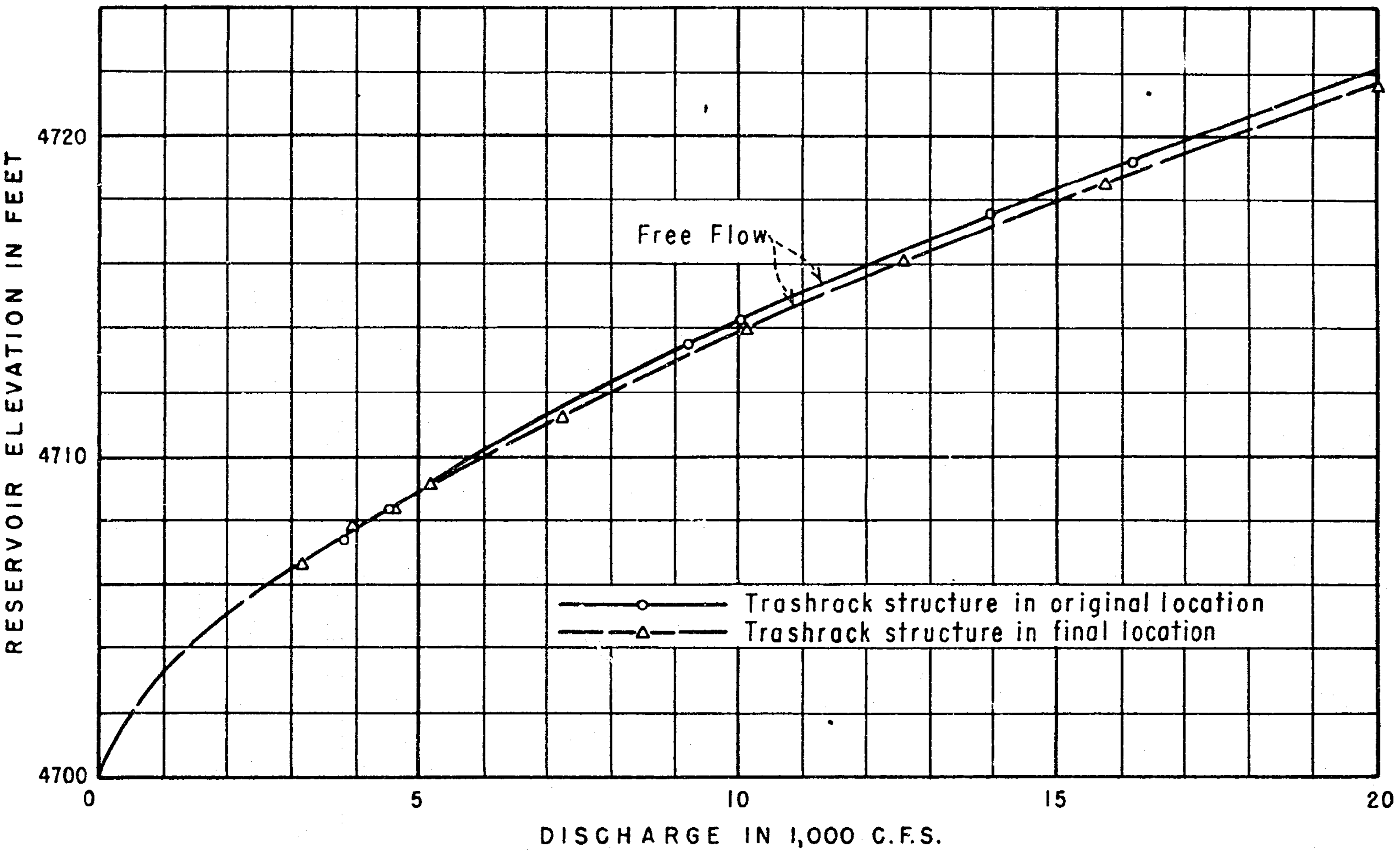


(a) Trashrack structure in original location.



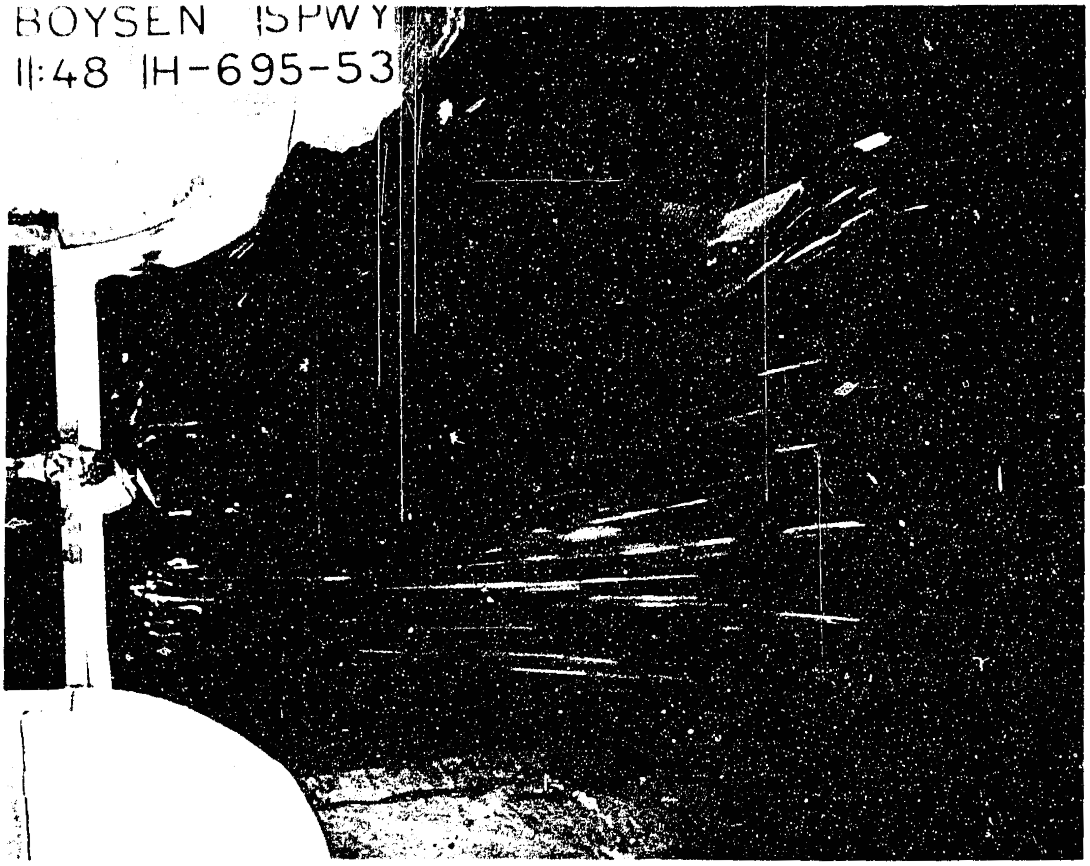
(b) Trashrack structure in final location.

BOYSEN DAM SPILLWAY
Flow In The Spillway Approach
Discharge 20,000 Second-feet And Gates Open 16 feet
1:48 Model



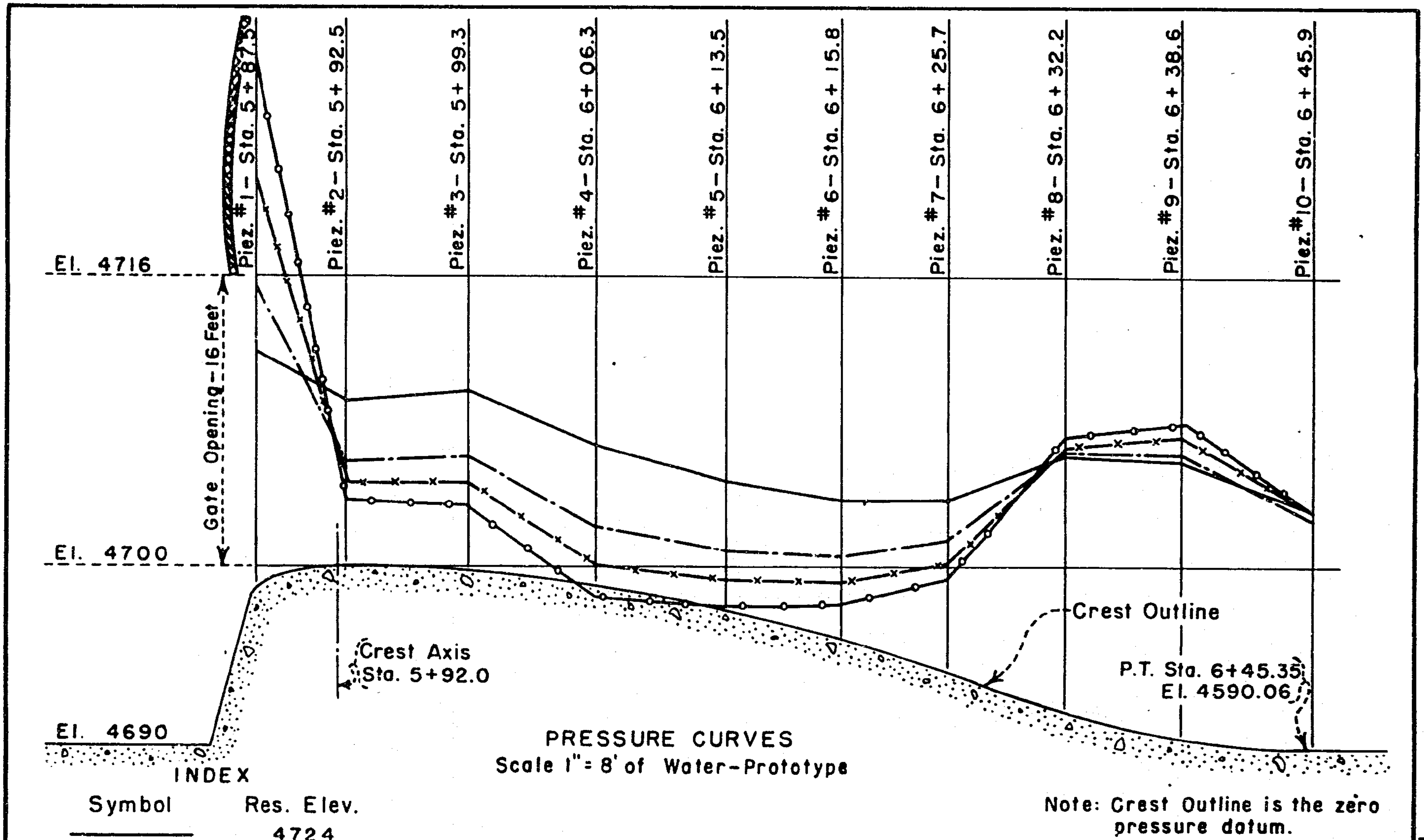
BOYSEN DAM SPILLWAY
DISCHARGE CAPACITY CURVES
1:48 MODEL

FIGURE 11



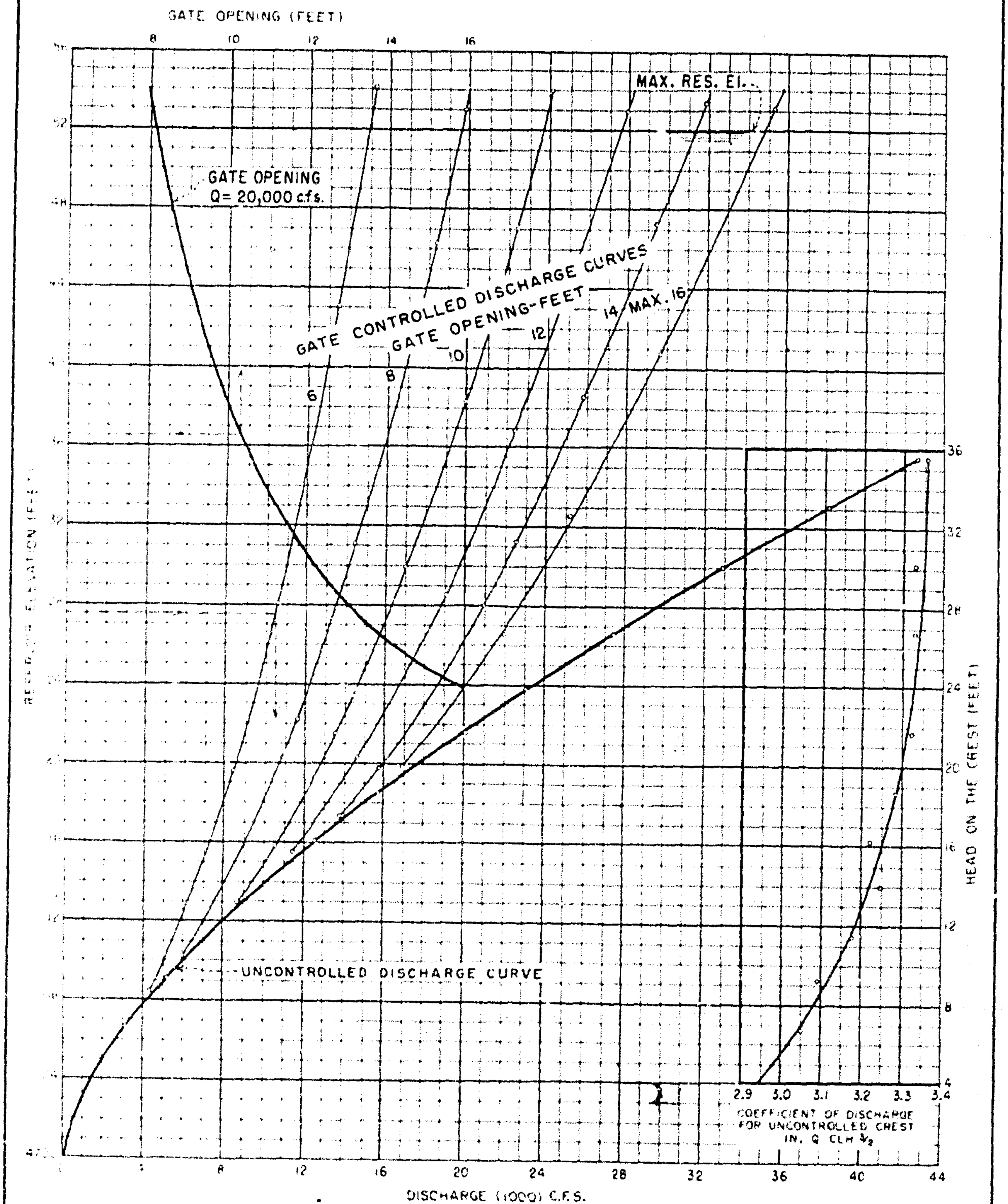
Trashrack in final location. Discharge 20,000 second-feet. Gates open 16 feet. (Photo is a one-half second exposure)

BOYSEN DAM SPILLWAY
Flow Lines In The Spillway Approach
1:48 Model



BOYSEN DAM SPILLWAY
CREST PRESSURES
DISCHARGE 20,000 SECOND FEET
1 : 48 MODEL

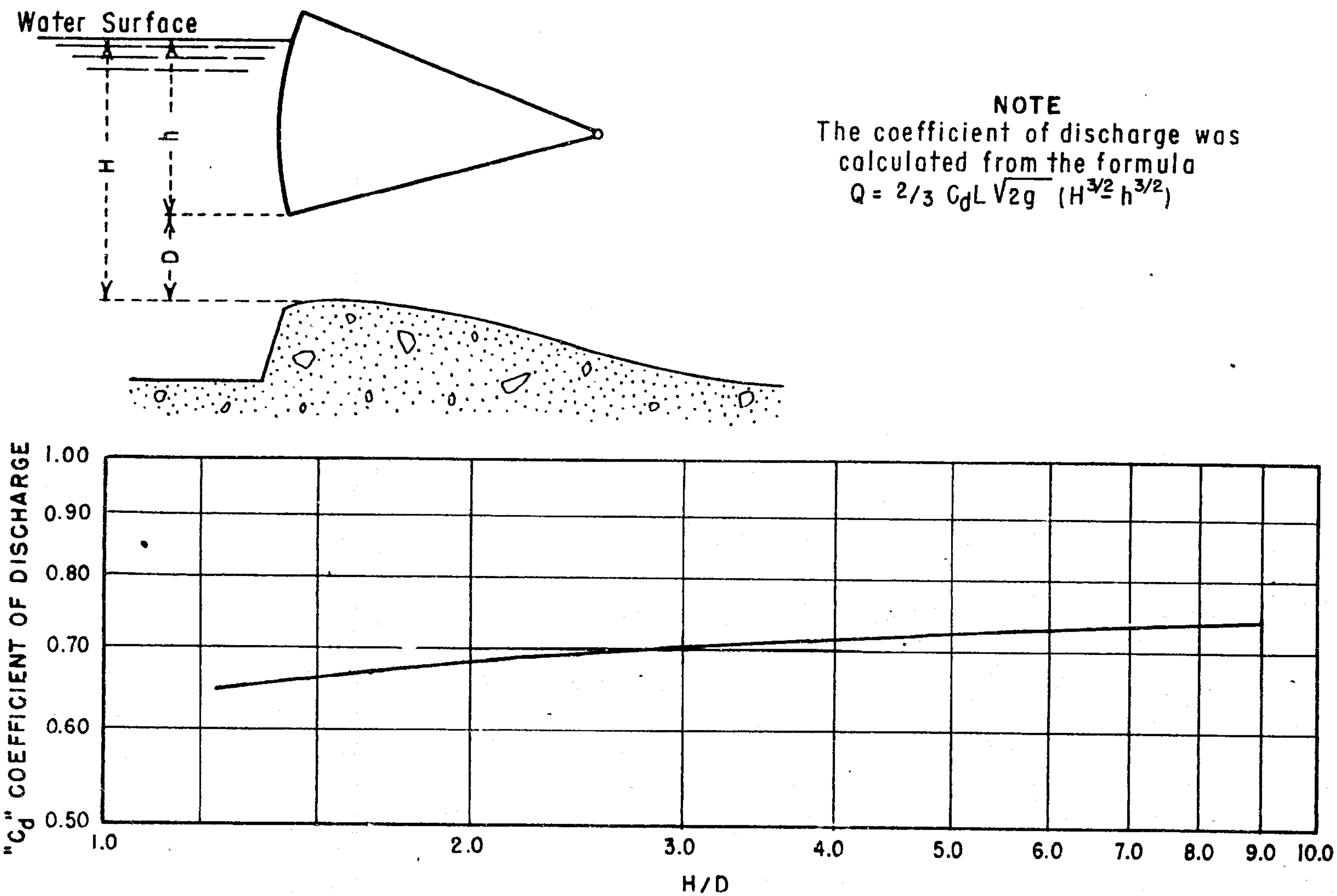
FIGURE 13



BOYSEN DAM SPILLWAY

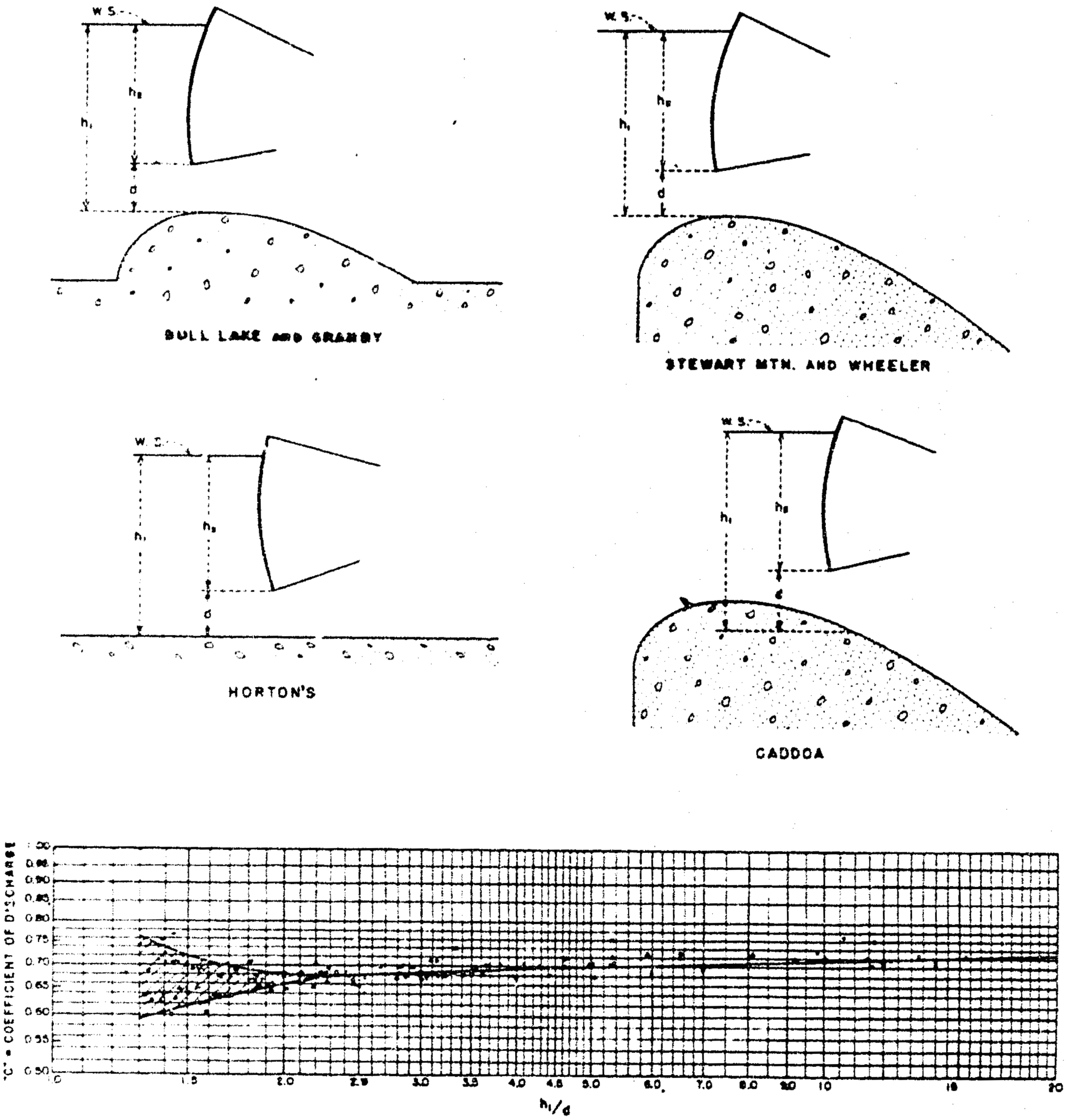
SPILLWAY DISCHARGE AND COEFFICIENT OF DISCHARGE CURVES

1:48 MODEL



BOYSEN DAM SPILLWAY
COEFFICIENT OF DISCHARGE CURVE
FOR THE CONTROLLED CREST
1:48 MODEL

FIGURE 16



NOTE: The coefficient of discharge was calculated from the formula $Q = \frac{2}{3} CL \sqrt{2g} \left(h_1^{3/2} - h_2^{3/2}\right)$.

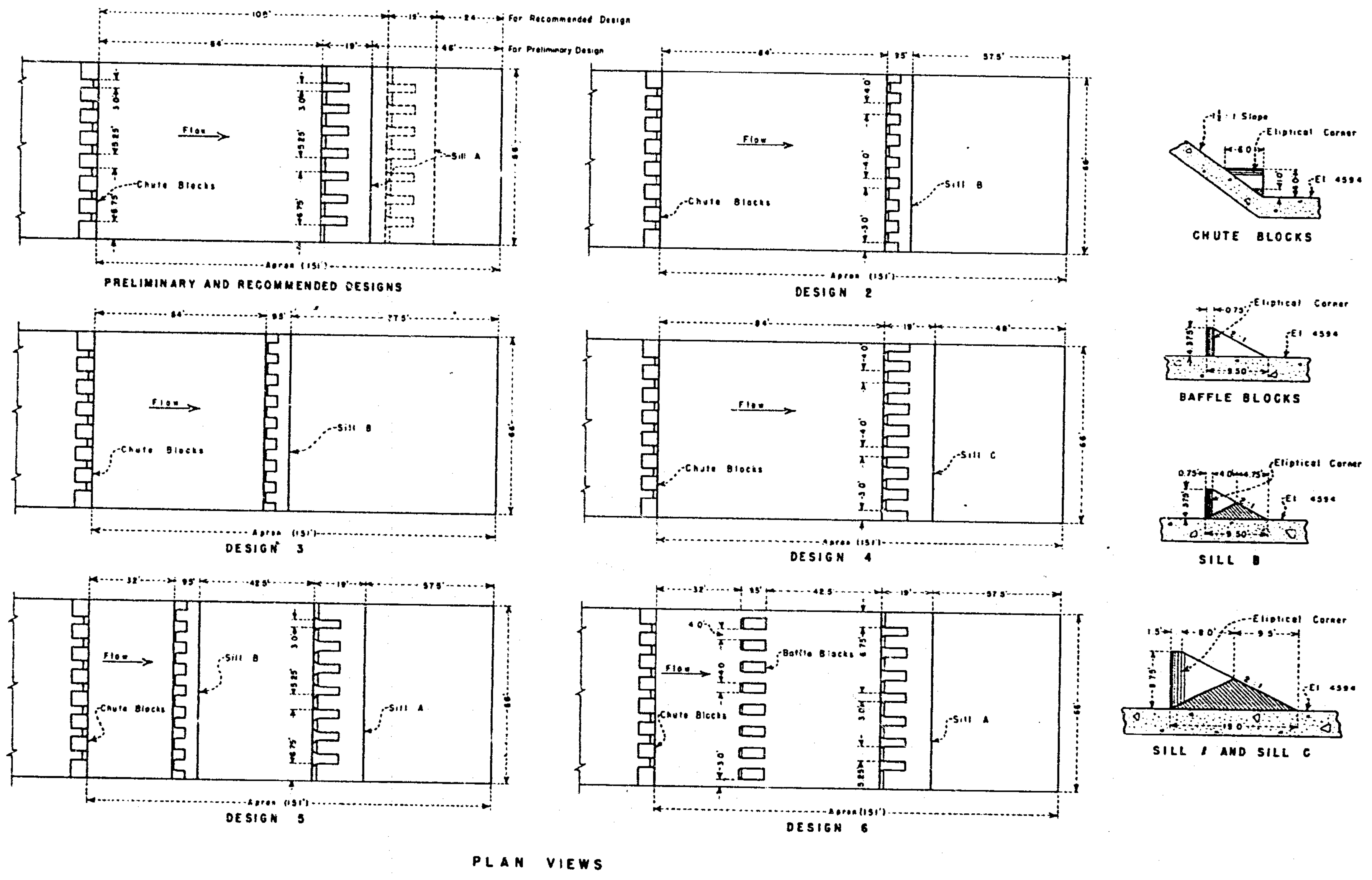
- Stewart Mountain Dam *Scale 1:50*
- Bull Lake Dam *Scale 1:30*
- Wheeler Dam *Scale 1:36*
- Caddoa Dam *Scale 1:36*
- R. E. Horton's Experiments
- Granby Dam. *Scale 1:48*

COEFFICIENT OF DISCHARGE
RADIAL GATES

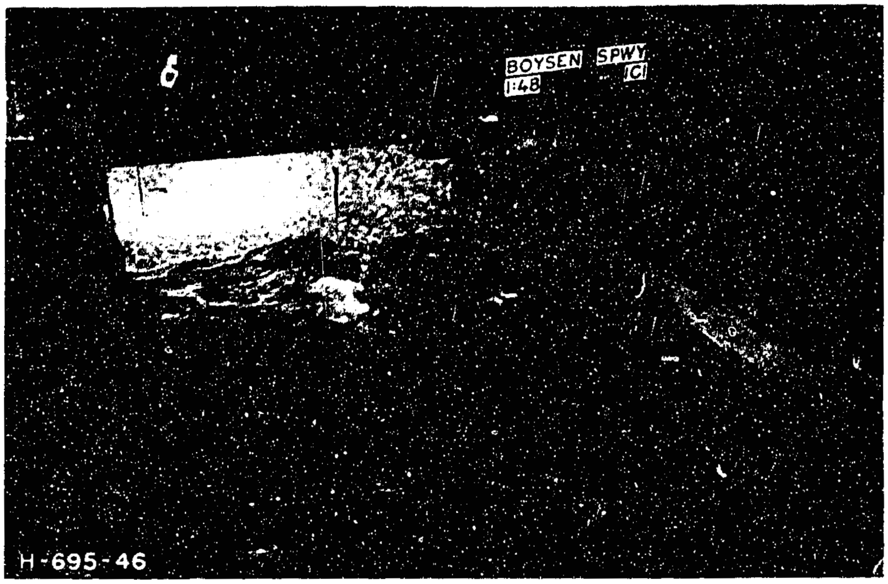
4-2-42 R.R.P.

X-D-441

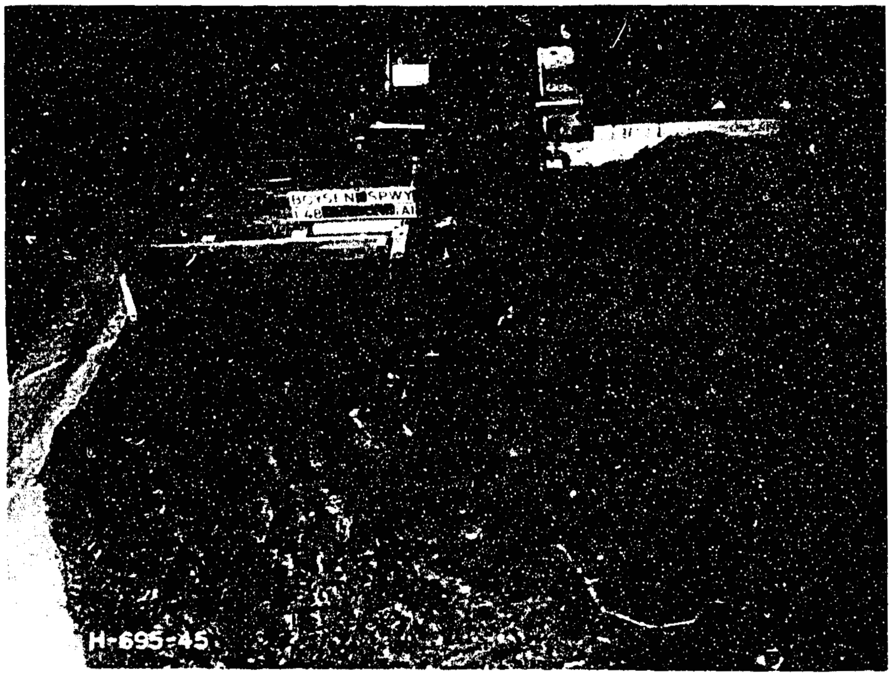
FIGURE 17



BOYSEN DAM SPILLWAY
STILLING BASIN DESIGNS
1 : 48 MODEL

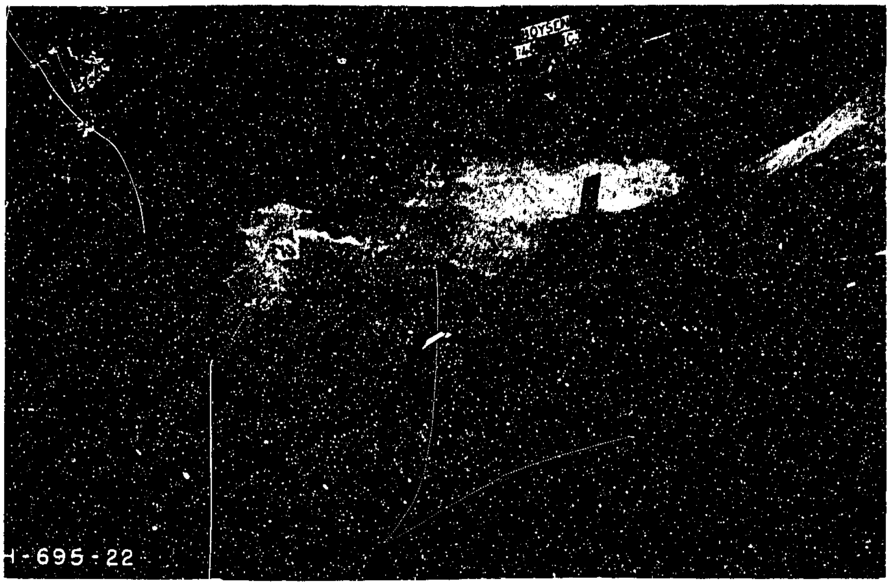


(a) Looking toward powerhouse. Right training wall extended 3-1/2 feet higher than preliminary design by use of the recommended sea-wall.

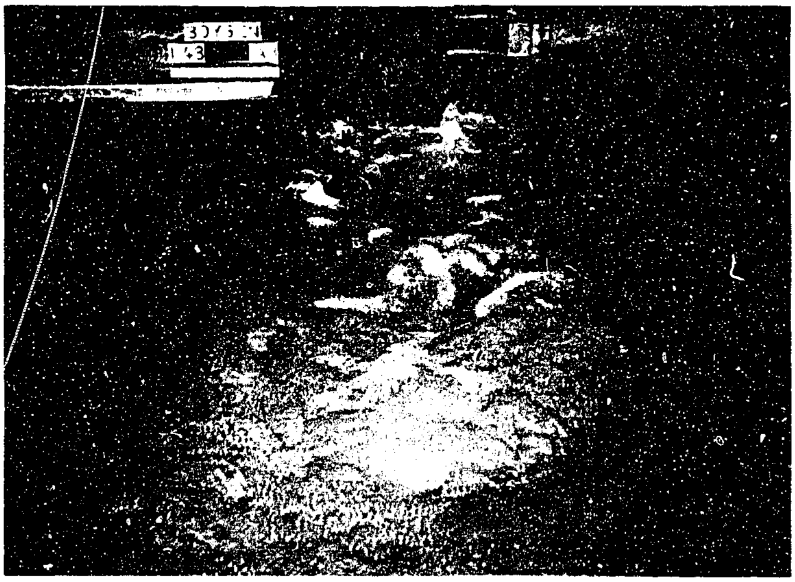


(b) Looking upstream.

BOYSEN DAM SPILLWAY
Flow In The Preliminary Stilling Basin Design
20,000 Second-feet
1:48 Model

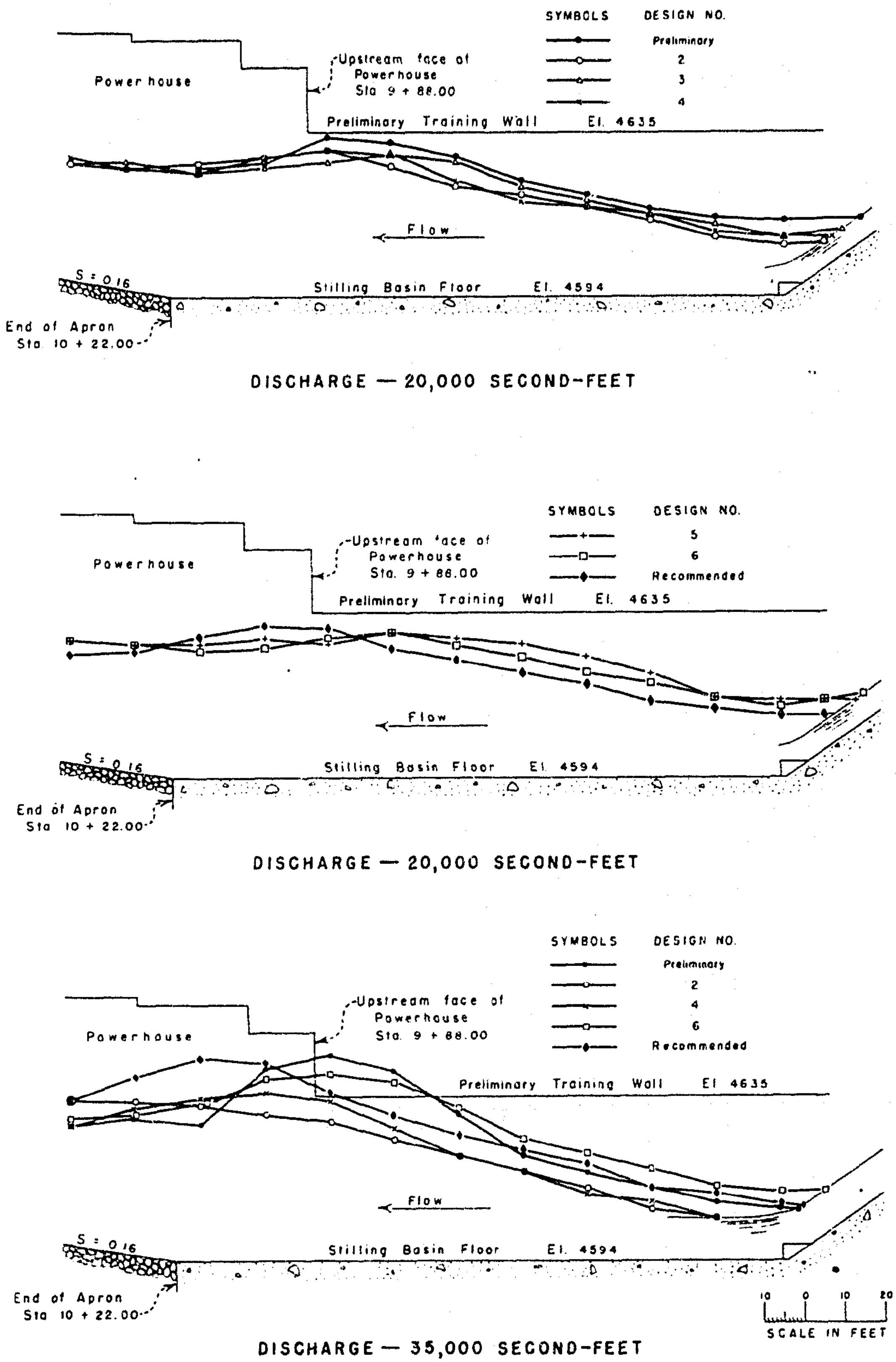


(a) Looking toward powerhouse. Right training wall extended 16 feet higher than preliminary design.



(b) Looking upstream.

BOYSEN DAM SPILLWAY
Flow In The Preliminary Stilling Basin Design
35,000 Second-feet
1:48 Model

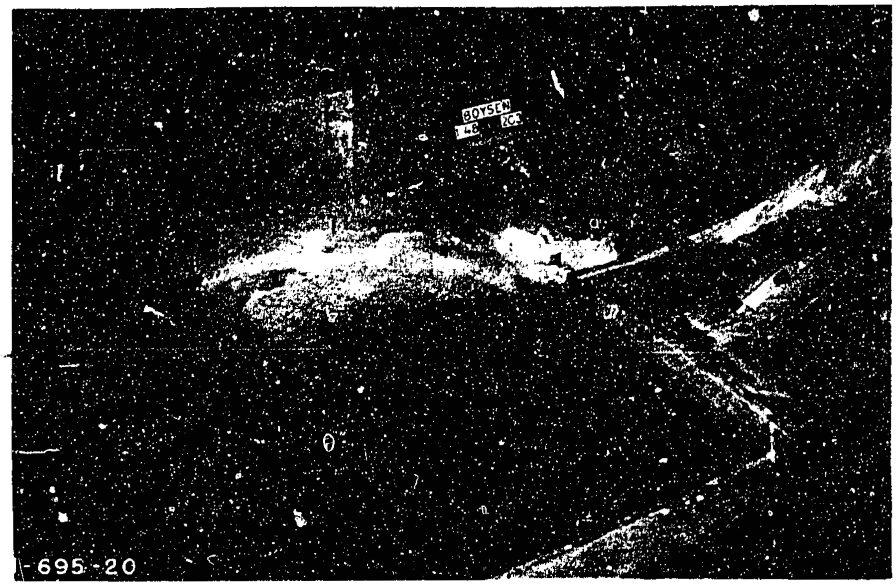


BOYSEN DAM SPILLWAY
WATER SURFACE PROFILES
1 : 48 MODEL

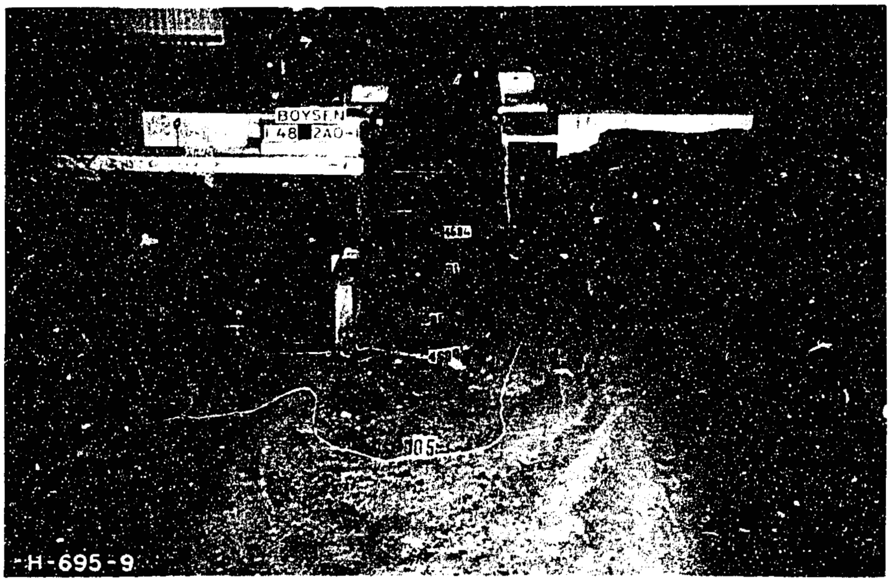
(a) Scour pattern after operating the model for 30 minutes. Discharge - 20,000 second-feet.

(b) Scour pattern after operating the model for 30 minutes. Discharge - 35,000 second-feet.

BOYSEN DAM SPILLWAY
Scour Patterns - Preliminary Stilling Basin Design
1:48 Model

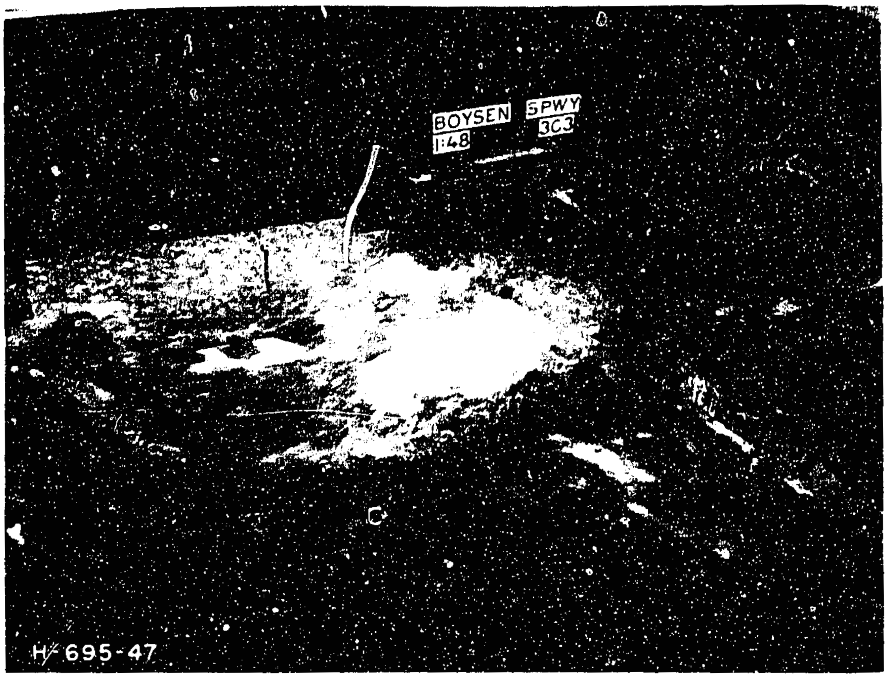


(a) Looking toward powerhouse. Right training wall extended 16 feet higher than the preliminary design. Discharge - 35,000 second-feet.

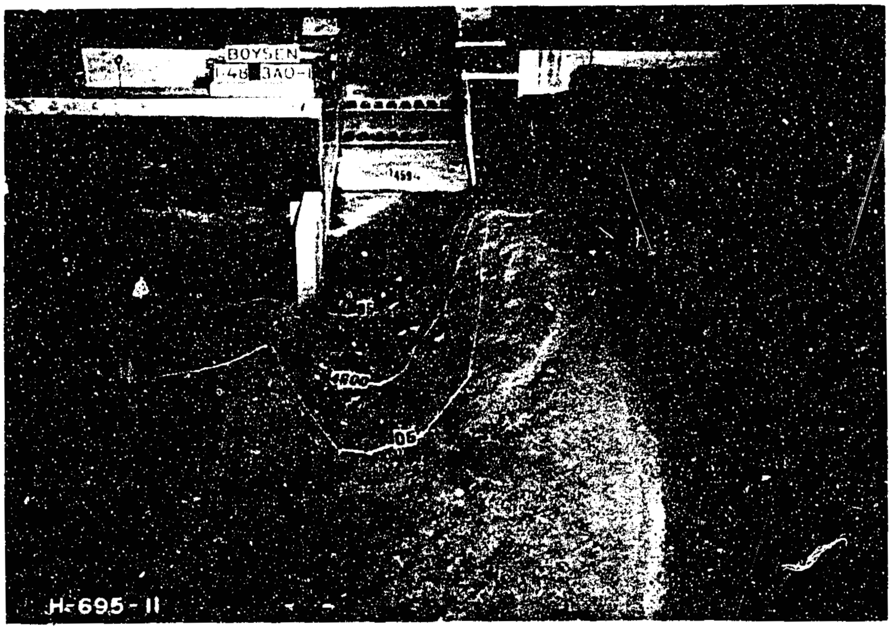


(b) Scour pattern after operating the spillway for 30 minutes. Discharge - 20,000 second-feet.

BOYSEN DAM SPILLWAY
Flow In Stilling Basin Design 2 And Scour Pattern
1:48 Model

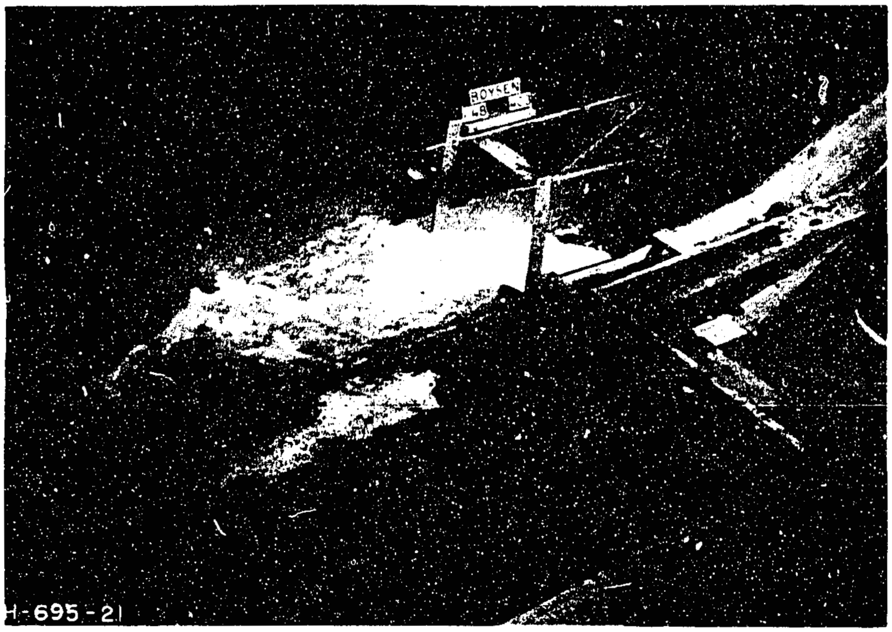


(a) Looking toward powerhouse. Right training wall extended 3-1/2 feet higher than preliminary design by use of recommended sea-wall. Discharge - 35,000 second-feet.



(b) Scour pattern after operating the model for 30 minutes. Discharge - 20,000 second-feet.

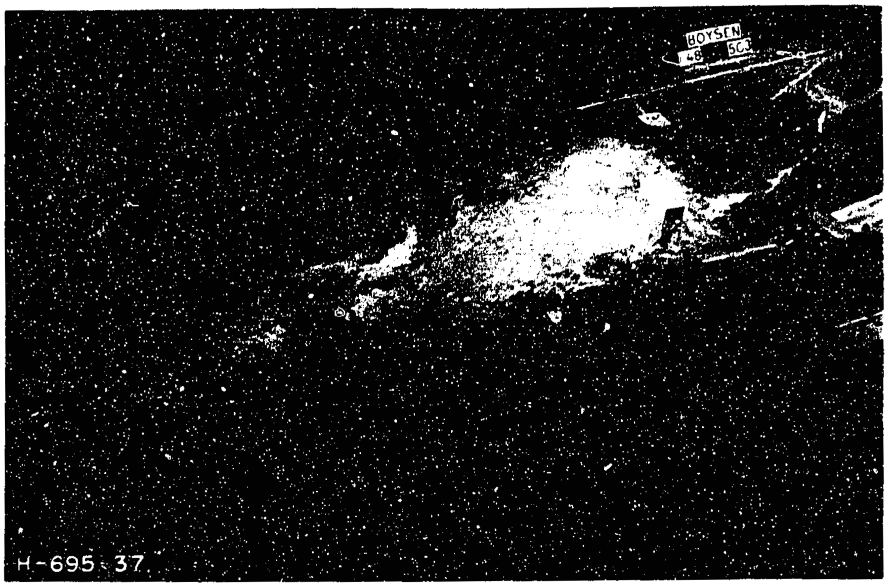
BOYSEN DAM SPILLWAY
Flow In Stilling Basin Design 3 and Scour Pattern
1:48 Model



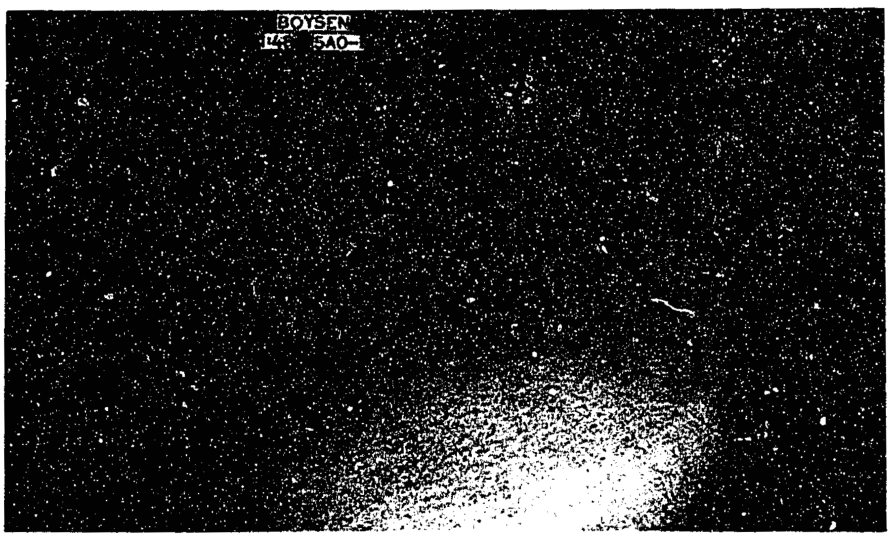
(a) Looking toward powerhouse. Right training wall extended 16 feet higher than the preliminary design. Discharge - 35,000 second-feet.

(b) Scour pattern after operating the model for 30 minutes. Discharge - 20,000 second-feet.

BOYSEN DAM SPILLWAY
Flow In Stilling Basin Design 4 and Scour Pattern
1:48 Model

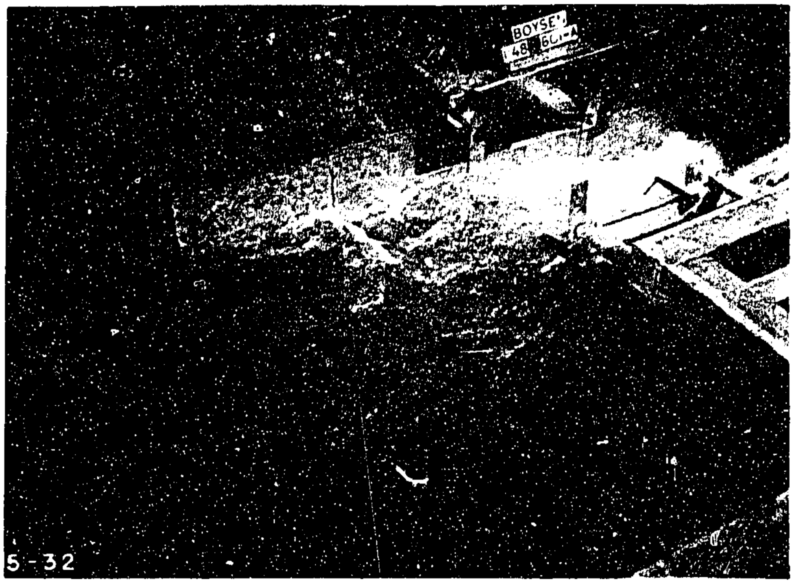


(a) Looking toward powerhouse. Right training wall extended 16 feet higher than the preliminary design. Discharge - 35,000 second-feet.

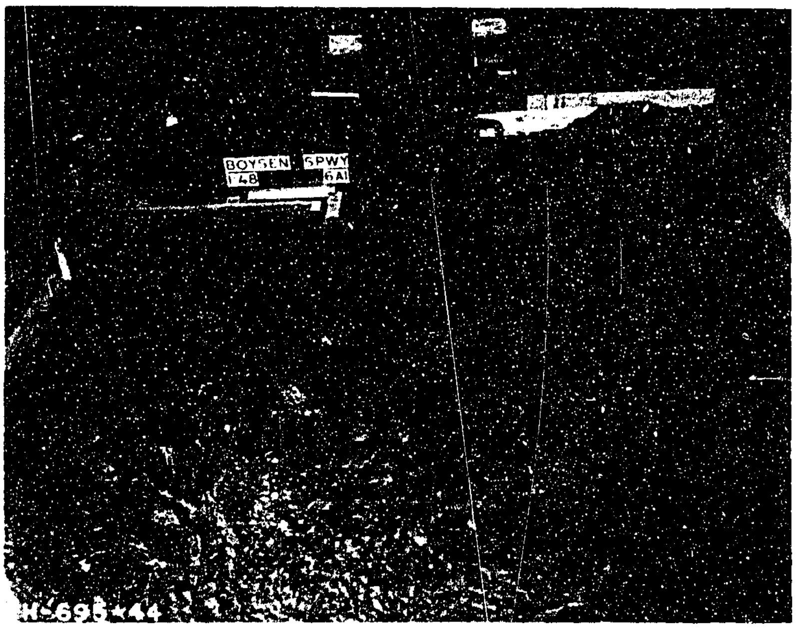


(b) Scour pattern after operating the model for 30 minutes. Discharge - 20,000 second-feet.

BOYSEN DAM SPILLWAY
Flow In Stilling Basin Design 5 and Scour Pattern
1:48 Model

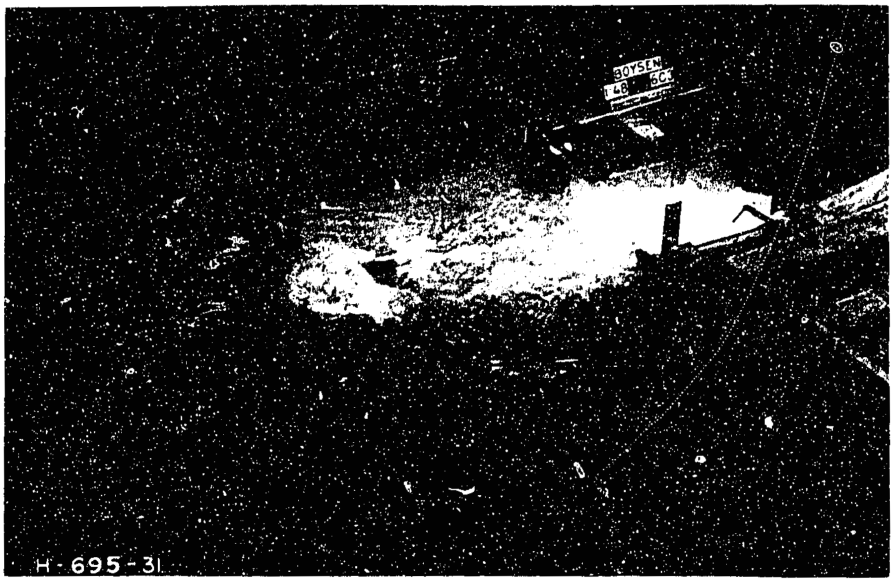


(a) Looking toward powerhouse. Right training wall extended 16 feet higher than prliminary design.



(b) Looking upstream.

BOYSEN DAM SPILLWAY
Flow In Stilling Basin Design 6 - 20,000 Second-feet
1:48 Model



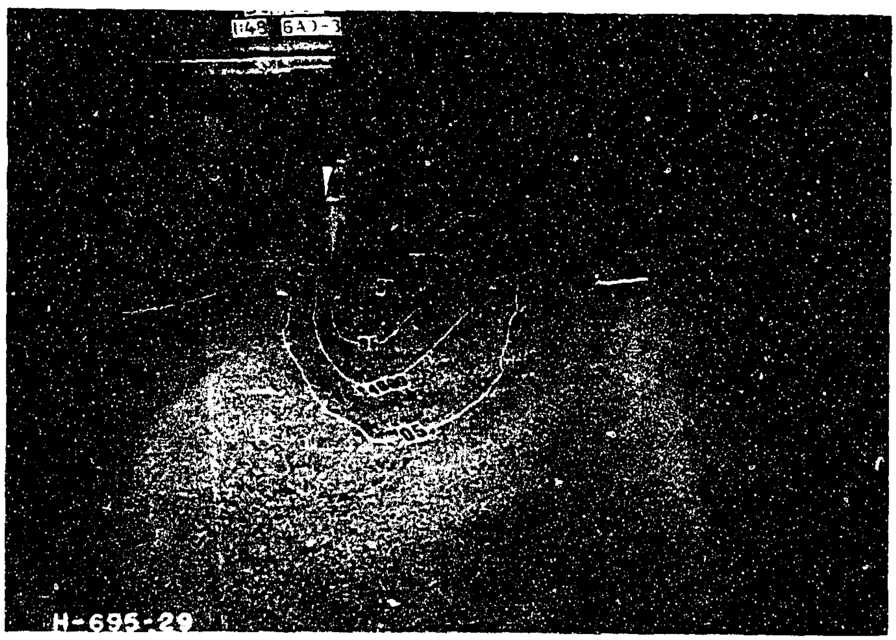
(a) Looking toward powerhouse. Right training wall extended 16 feet higher than preliminary design.

(b) Looking upstream.

BOYSEN DAM SPILLWAY
Flow In Stilling Basin Design 6 - 35,000 Second-feet
1:48 Model

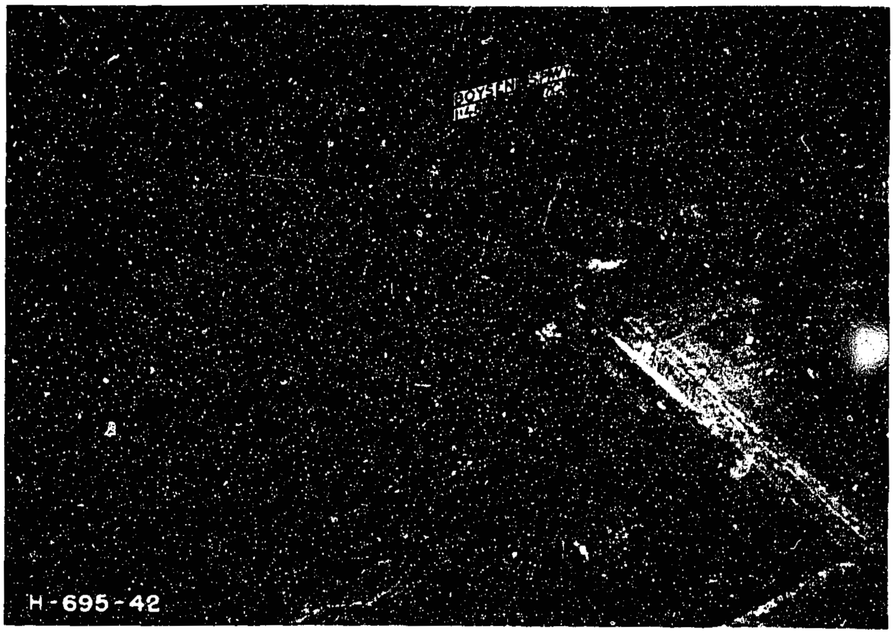
Figure 28

(a) Scour pattern after operating the model for 30 minutes. Discharge - 20,000 Second-feet.

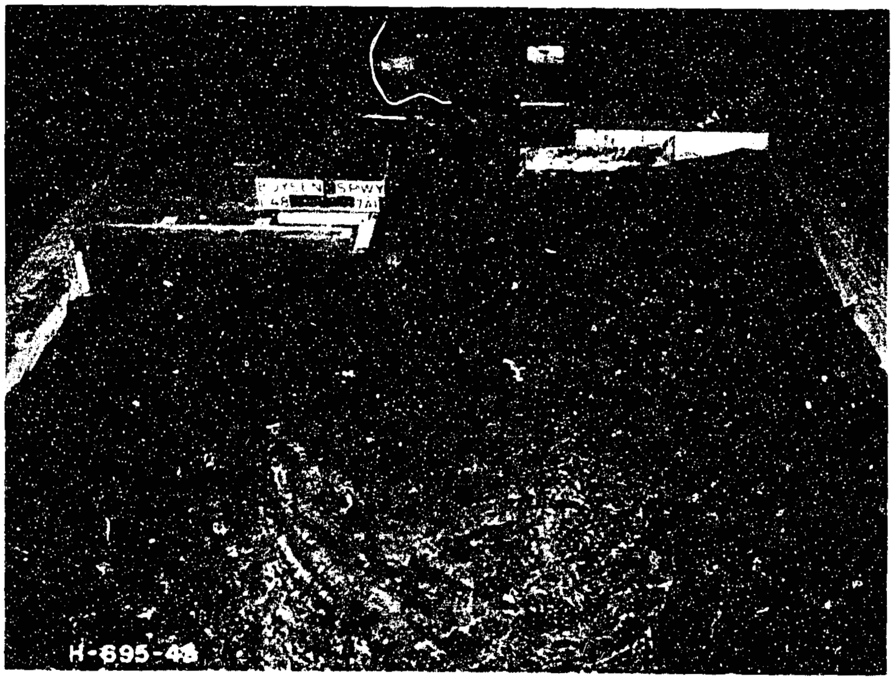


(b) Scour pattern after operating the model for 30 minutes. Discharge - 35,000 Second-feet.

BOYSEN DAM SPILLWAY
Scour Patterns - Stilling Basin Design 6
1:48 Model

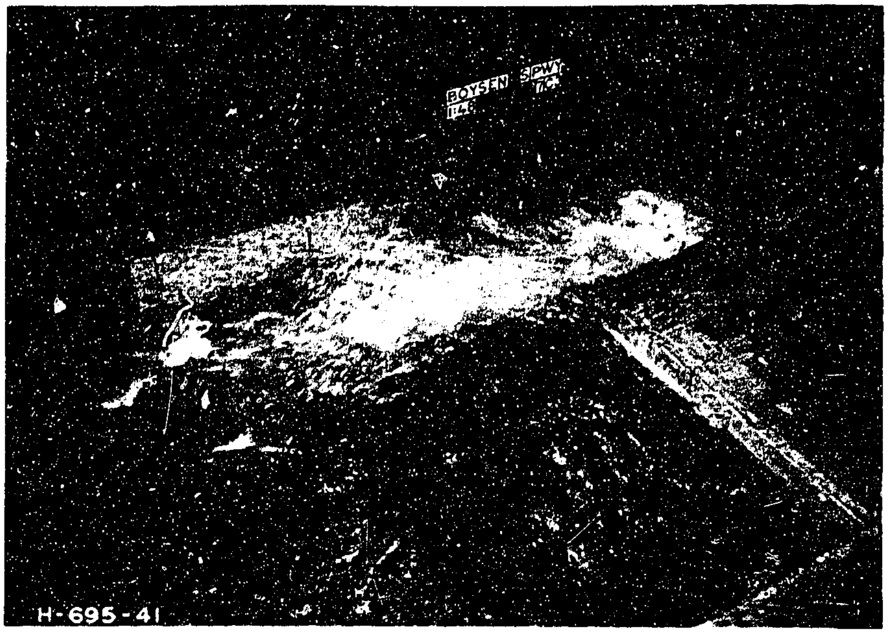


(a) Looking toward powerhouse. Right training wall extended 3-1/2 feet higher than preliminary design by use of recommended sea-wall.

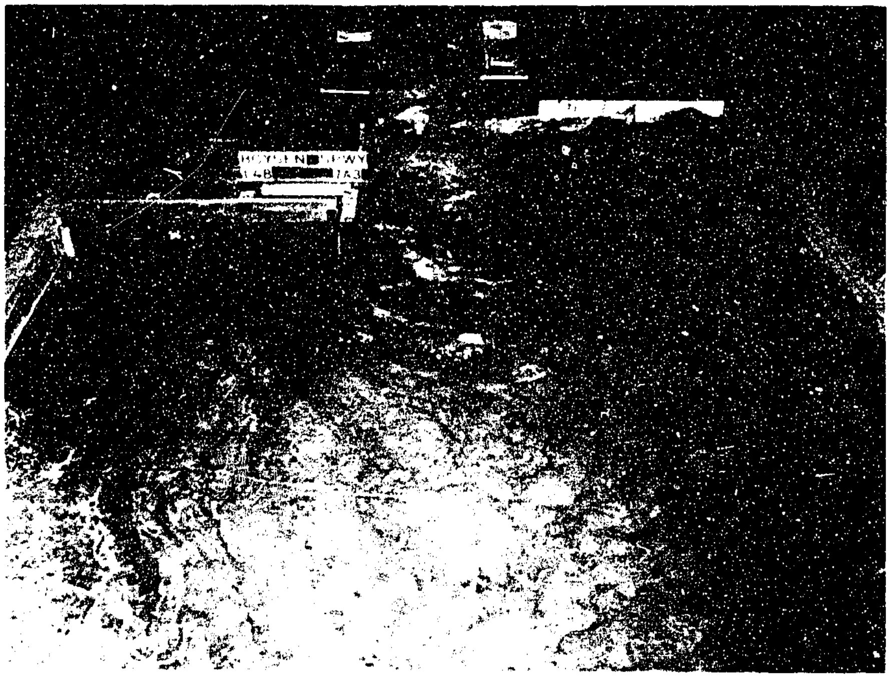


(b) Looking upstream.

BOYSEN DAM SPILLWAY
Flow In The Recommended Stilling Basin Design
20,000 Second-feet
1:48 Model

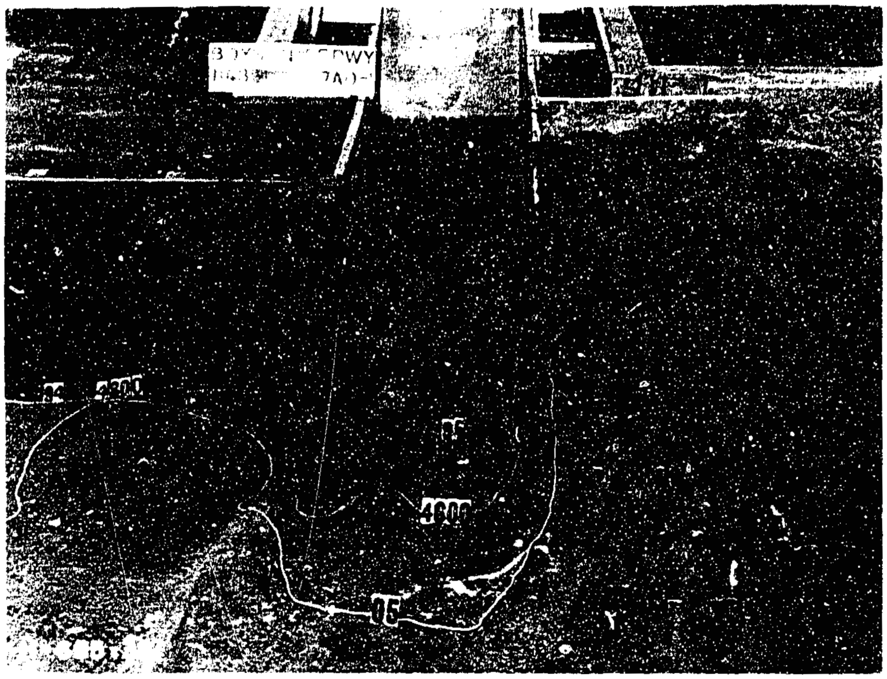


(a) Looking toward powerhouse. Right training wall extended 3-1/2 feet higher than the preliminary design by use of recommended sea-wall.



(b) Looking upstream.

BOYSEN DAM SPILLWAY
Flow In The Recommended Stilling Basin Design
35,000 Second-feet
1:48 Model



(a) Scour pattern after operating the model 30 minutes. Discharge - 20,000 Second-feet.

(b) Scour pattern after operating the model 30 minutes. Discharge - 35,000 Second-feet.

BOYSEN DAM SPILLWAY
Scour Patterns - Recommended Stilling Basin Design
1:48 Model

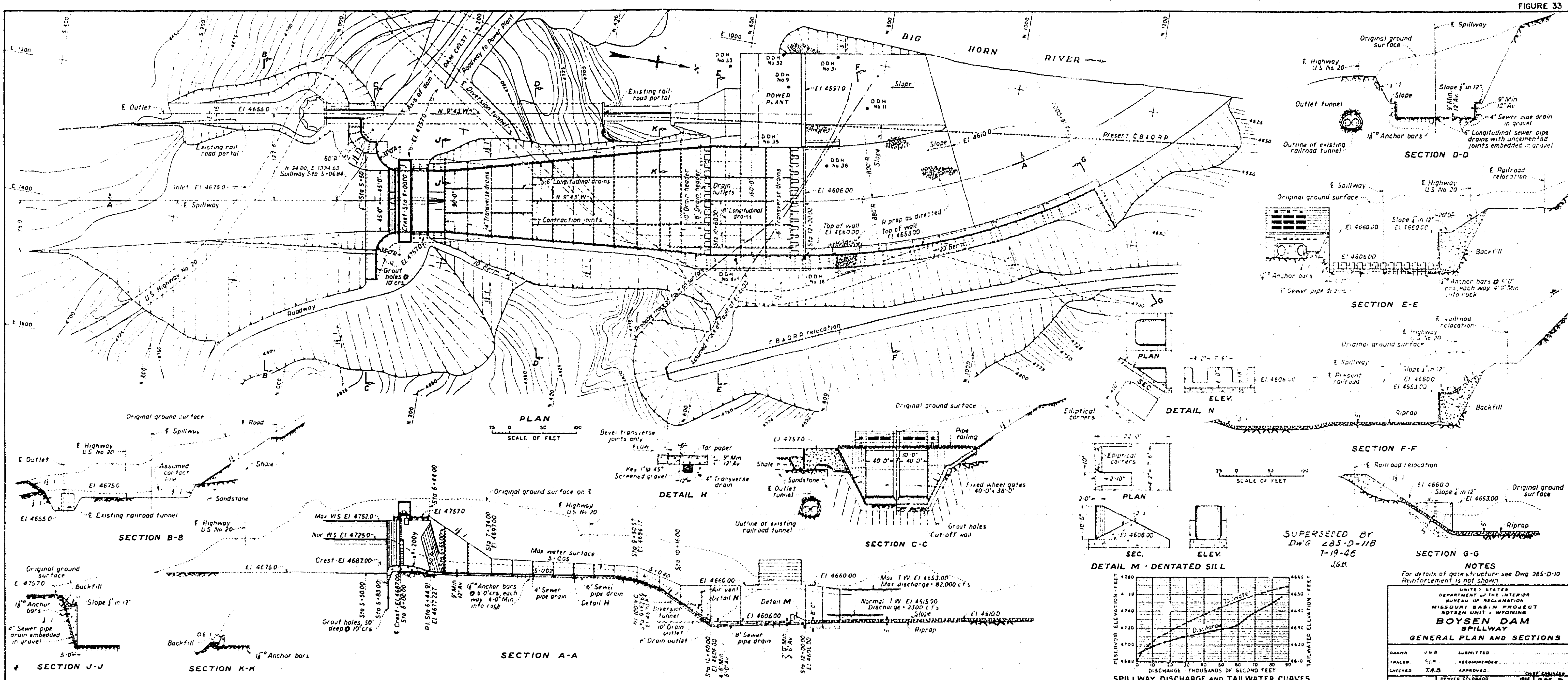
DESIGN	DISCHARGE-20,000 SECOND-FEET		DISCHARGE-35,000 SECOND-FEET				
	MAX. ELEV. OF AVERAGE WATER SURFACE PROFILE ALONG RIGHT TRAINING WALL*	ELEV. OF MAX. SCOUR DEPTH	MAX. ELEV. OF AVERAGE WATER SURFACE PROFILE ALONG RIGHT TRAINING WALL*	ELEV. OF MAX. SCOUR DEPTH**	TAILWATER ELEV. AT WHICH JUMP IS SWEPT FROM THE APRON***	TAILWATER ELEV. REQUIRED TO BRING JUMP BACK ON THE APRON***	DESCRIPTION OF LARGE EDDY THAT OCCURS DOWNSTREAM AND TO THE LEFT OF STILLING BASIN
Preliminary	4634	4590	4645	4587	4629.4	4632.2	Distinct.
2	4630	4591	4635	—	4635.2	4636.0.	Very distinct.
3	4630	4586	—	—	4635.2	4636.1	Distinct.
4	4631	4588	4636	—	4632.2	4633.9	Distinct.
5	4629	4585	Jump is swept from the Apron at Normal Tailwater.				
6	4630	4589	4641	4589	4632.1	4637.6	Not distinct.
Recommended	4632	4590	4639	4587	4628.6	4630.4	Distinct.

* Top of the Preliminary Right Training Wall is at Elev. 4635.0.
** Spillway Apron is at Elev. 4594.0.
*** Normal expected tailwater Elev., with 35,000 second-feet, is 4635.4.

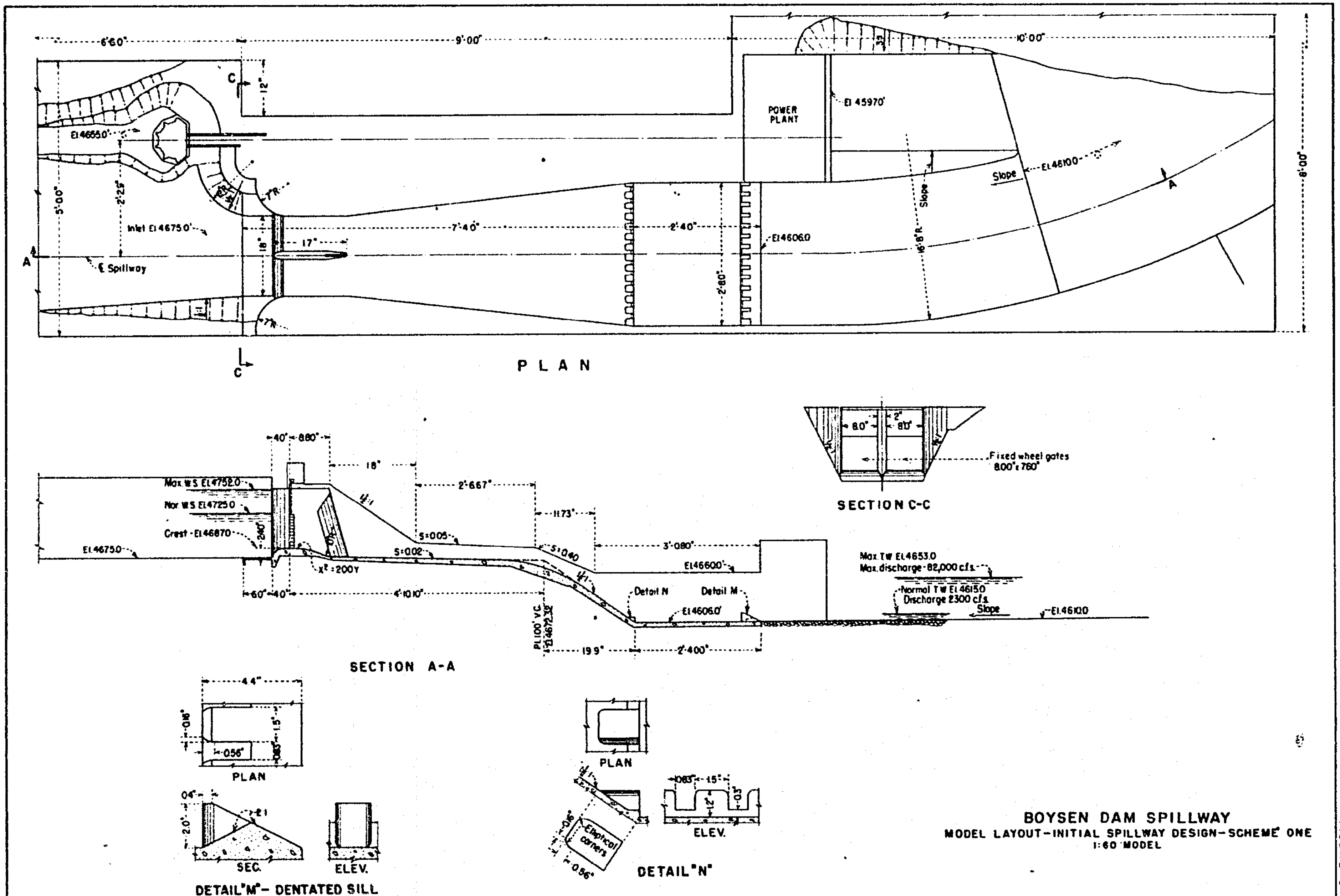
BOYSEN DAM SPILLWAY
SUMMARY OF MODEL TEST DATA ON ALL STILLING BASIN DESIGNS
1 : 48 MODEL

FIGURE 32

FIGURE 33

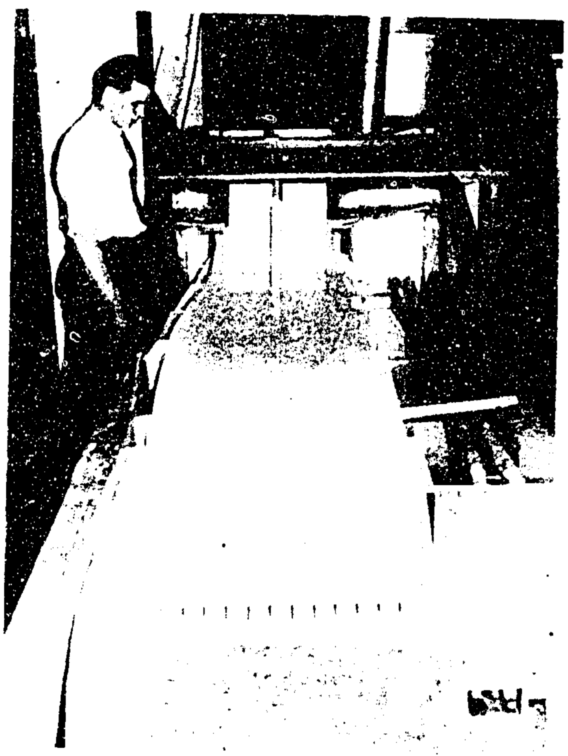


BIG HORN RIVER
POWER PLANT
PLAN
SCALE OF FEET
SECTION A-A
SECTION B-B
SECTION C-C
SECTION D-D
SECTION E-E
SECTION F-F
SECTION G-G
SECTION J-J
SECTION K-K
DETAIL H
DETAIL M - DENTATED SILL
DETAIL N
SPILLWAY DISCHARGE AND TAILWATER CURVES
DISCHARGE - THOUSANDS OF SECOND FEET
RESERVOIR ELEVATION - FEET
TAILWATER ELEVATION - FEET
Max WS El 4752.0
Nor WS El 4725.0
Crest El 4687.00
Max TW El 4653.00
Max discharge = 82,000 c.f.s.
Normal TW El 4615.90
Discharge = 2300 c.f.s.
Fixed wheel gates 40'-0" x 38'-0"
Outline of existing railroad tunnel
U.S. Highway No 20
C B & Q R R relocation
SUPERSEDED BY DWG 285-D-118 7-19-46 J.G.B.
NOTES
For details of gate structure see Dwg 285-D-10
Reinforcement is not shown
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MISSOURI BASIN PROJECT
BOYSEN UNIT - WYOMING
BOYSEN DAM
SPILLWAY
GENERAL PLAN AND SECTIONS
DENVER, COLORADO
285-D-8

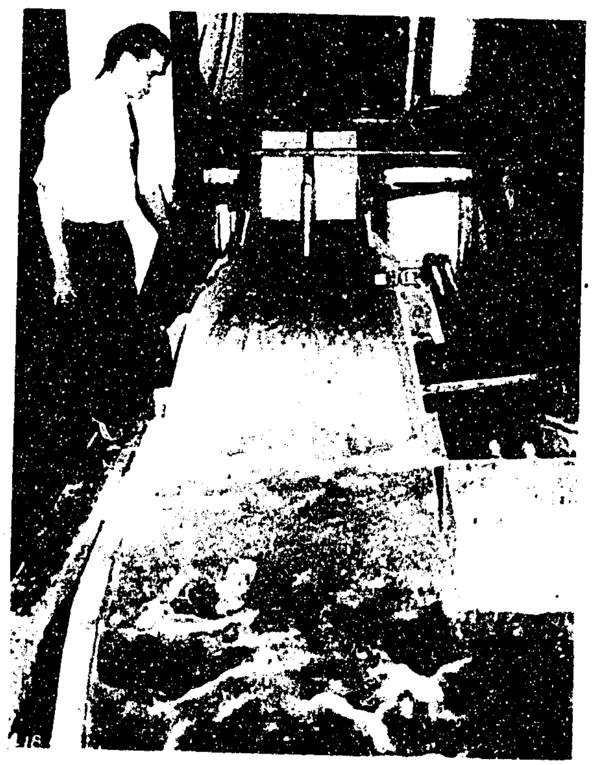


BOYSEN DAM SPILLWAY
MODEL LAYOUT-INITIAL SPILLWAY DESIGN-SCHEME ONE
1:60 MODEL

(a) Gate structure.

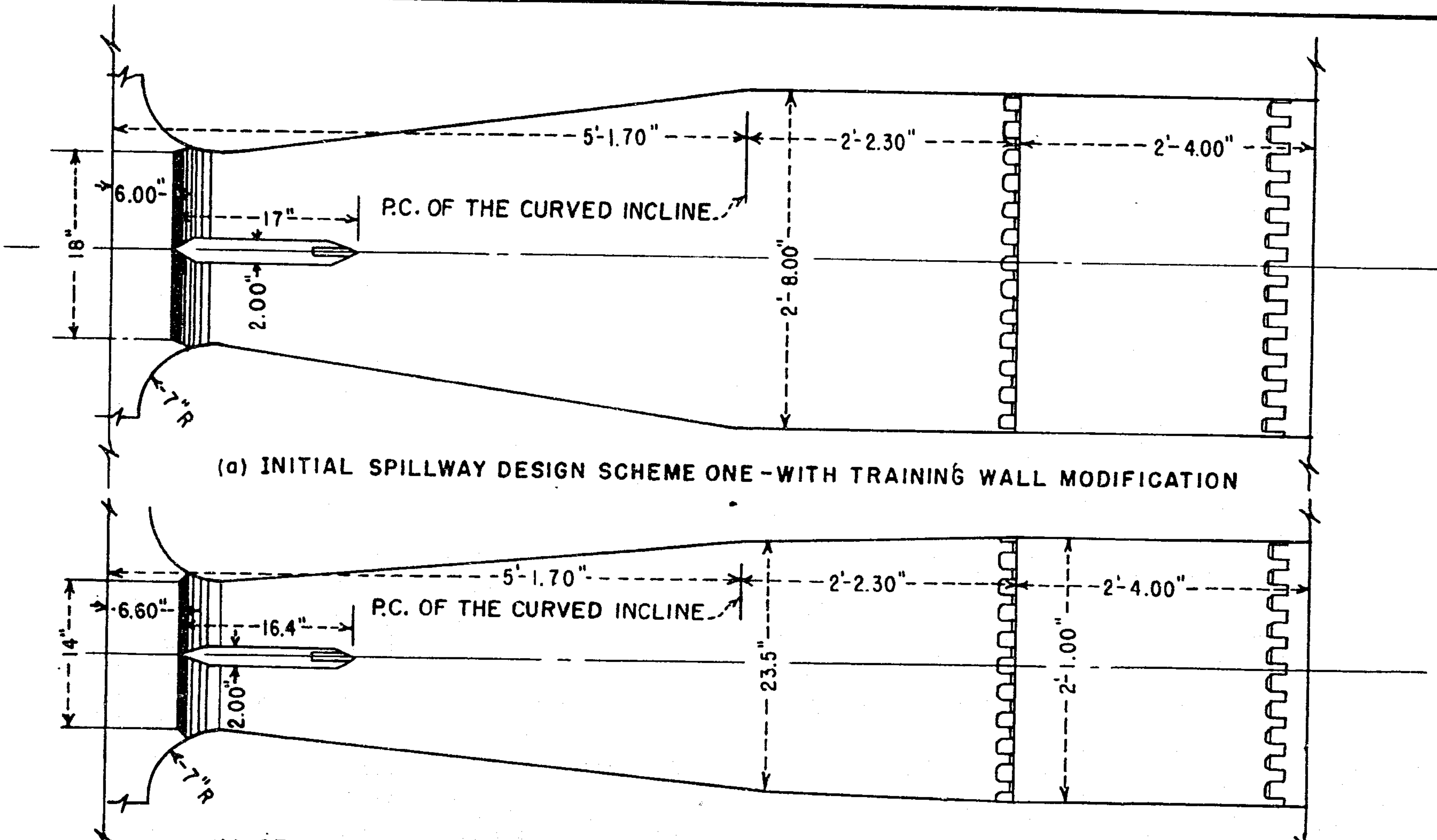


(b) Upstream view.



(c) Discharge - 82,000 second-feet.

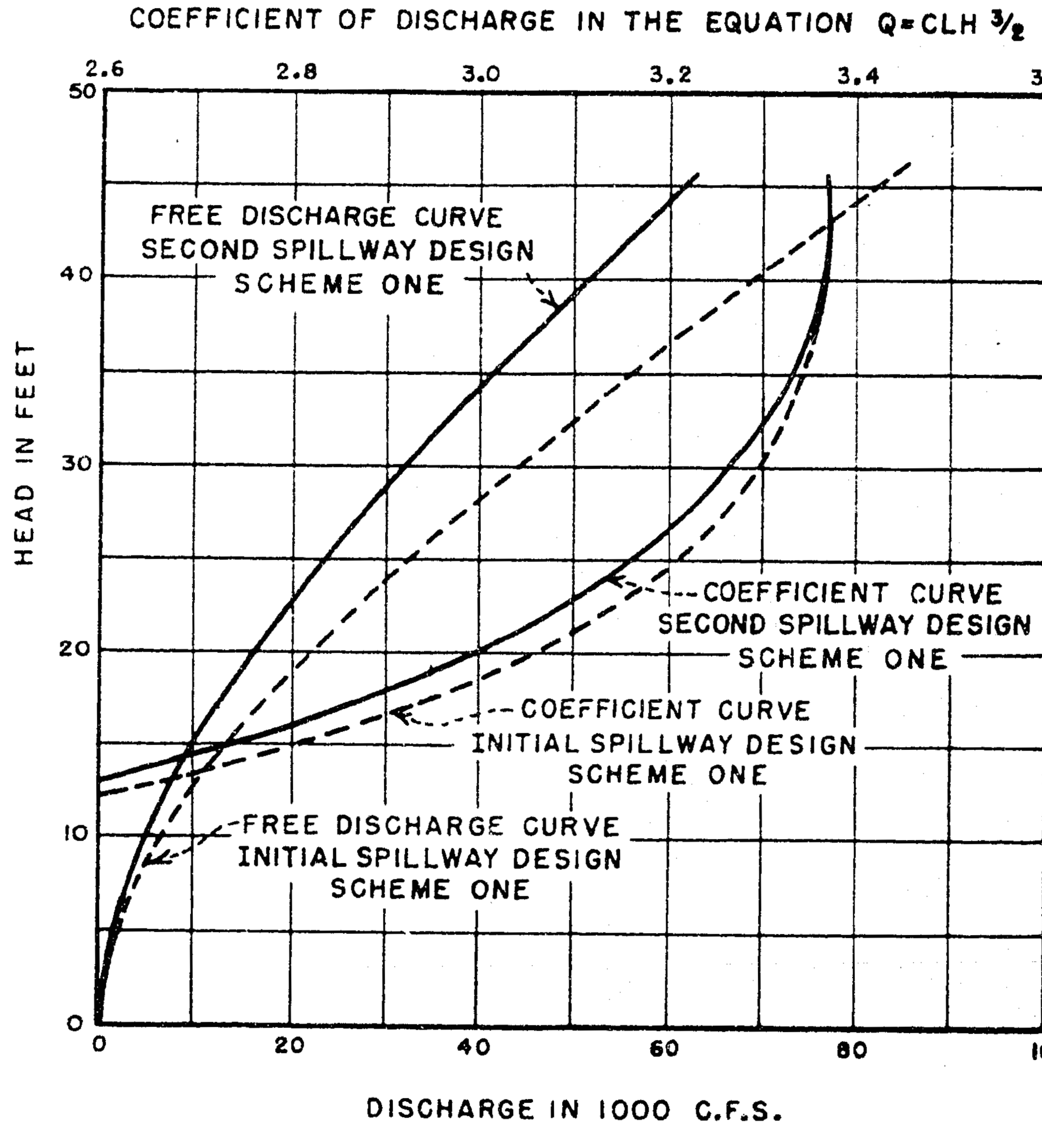
BOYSEN DAM SPILLWAY
Model Views - Initial Spillway Design - Scheme One
1:60 Model



(a) INITIAL SPILLWAY DESIGN SCHEME ONE - WITH TRAINING WALL MODIFICATION

(b) SECOND SPILLWAY DESIGN SCHEME ONE - WITH TRAINING WALL MODIFICATION

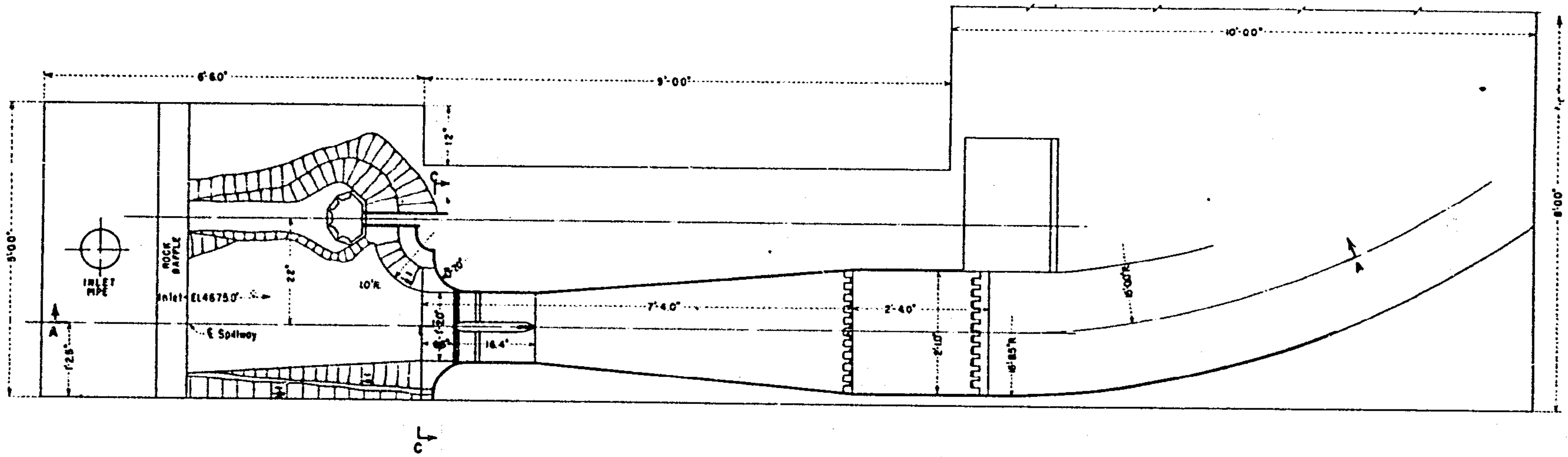
BOYSEN DAM SPILLWAY
TRAINING WALL MODIFICATIONS
1:60 MODEL

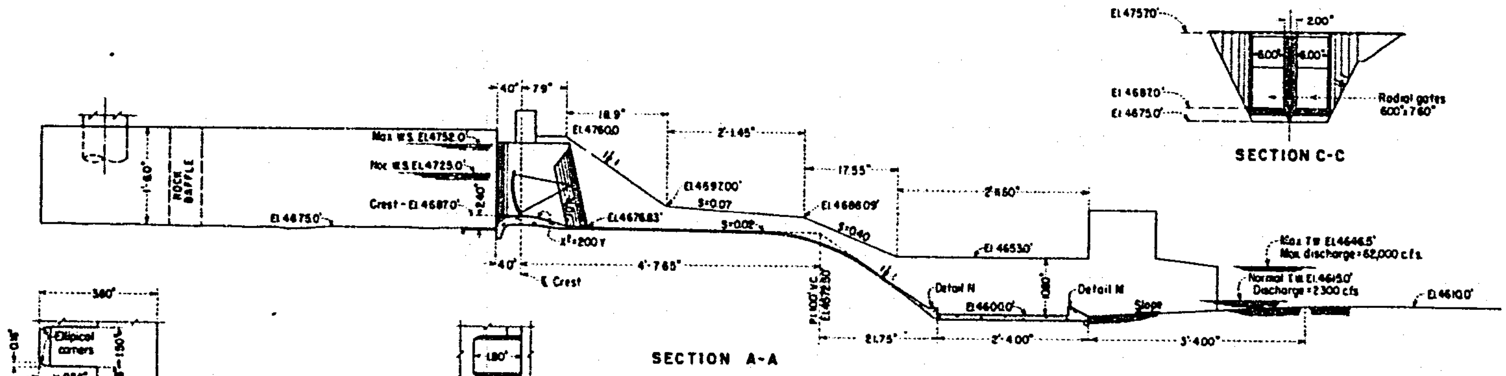


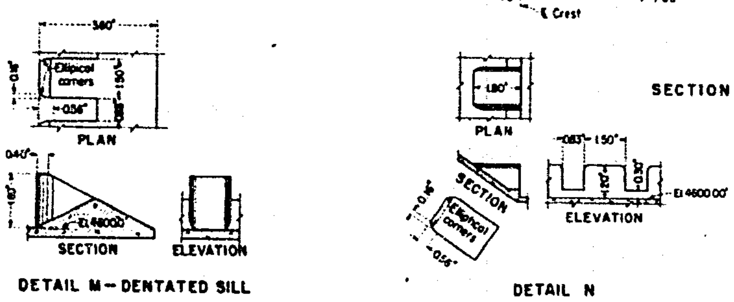
BOYSEN DAM SPILLWAY

CALIBRATION AND COEFFICIENT OF DISCHARGE CURVES

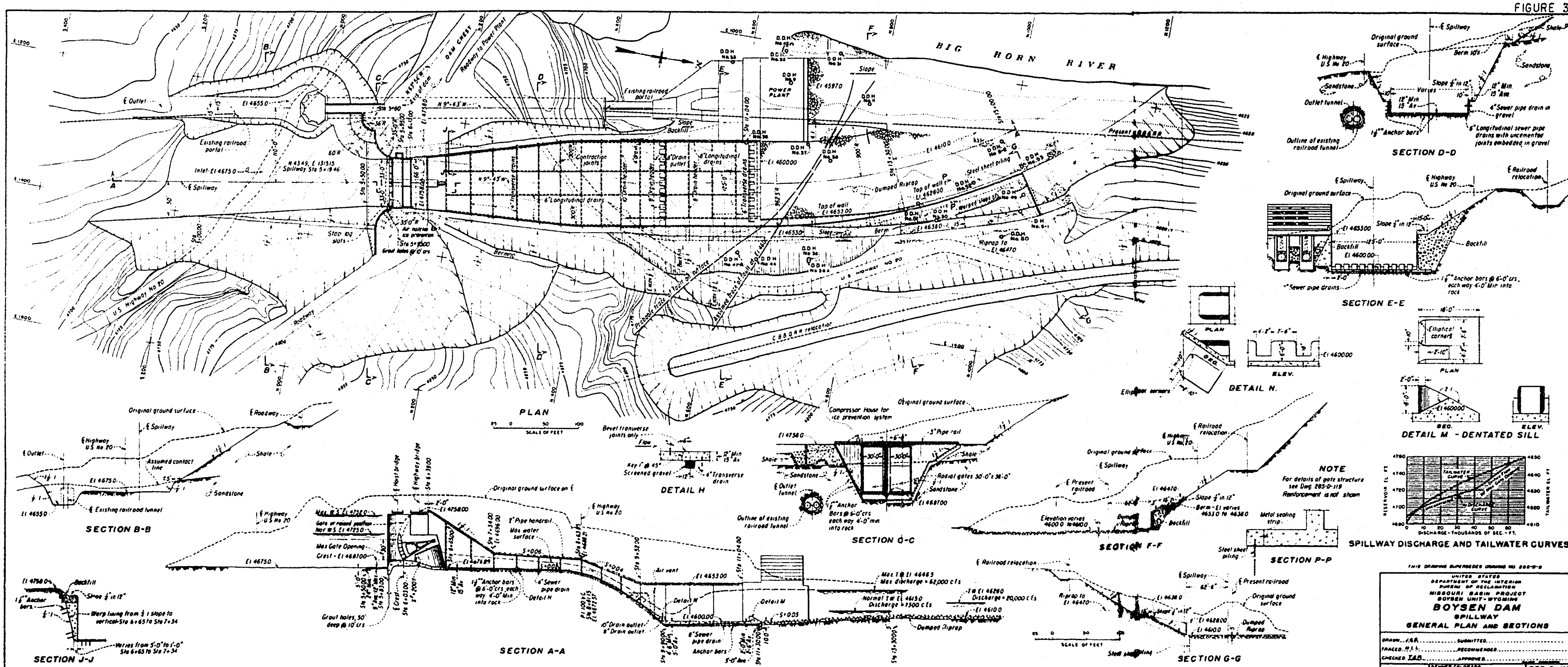
1:60 MODEL

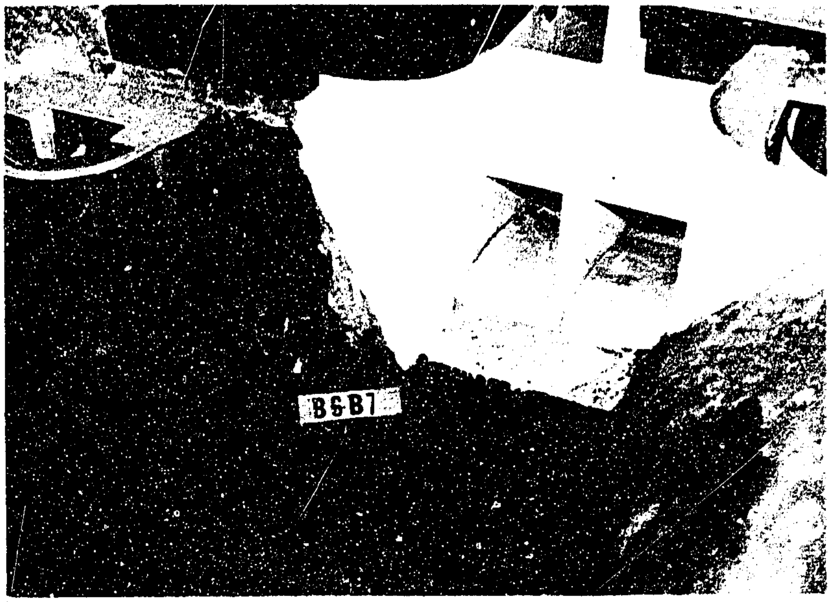




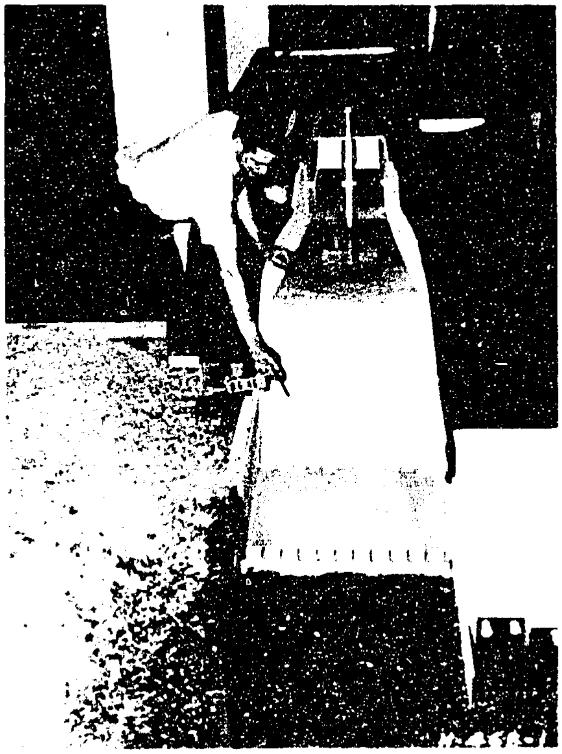


BOYSEN DAM SPILLWAY
MODEL LAYOUT—SECOND SPILLWAY DESIGN—SCHEME ONE
1:60 MODEL

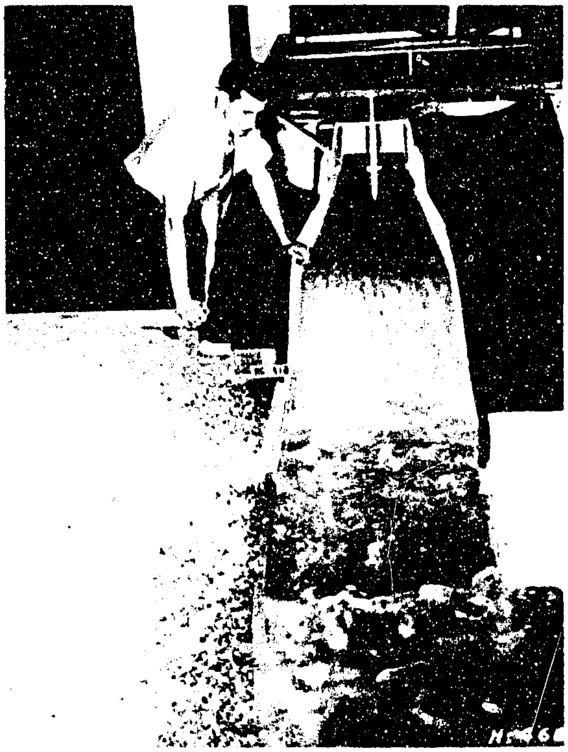

BIG HORN RIVER
PLAN
SCALE OF FEET
SECTION B-B
SECTION A-A
SECTION J-J
DETAIL H
SECTION C-C
SECTION D-D
SECTION E-E
DETAIL N.
DETAIL M - DENTATED SILL
SECTION F-F
SECTION P-P
SECTION G-G
NOTE
For details of gate structure see Dwg. 285-D-119
Reinforcement is not shown
DISCHARGE - THOUSANDS OF SEC.-FT.
SPILLWAY DISCHARGE AND TAILWATER CURVES
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MISSOURI BASIN PROJECT
BOYSEN UNIT - WYOMING
BOYSEN DAM
SPILLWAY
GENERAL PLAN AND SECTIONS
DRAWN J.G.R.
SUBMITTED
TRACED M.L.L.
RECOMMENDED
CHECKED T.A.B.
APPROVED
DENVER, COLORADO
285-D-118



(a) Gate structure.

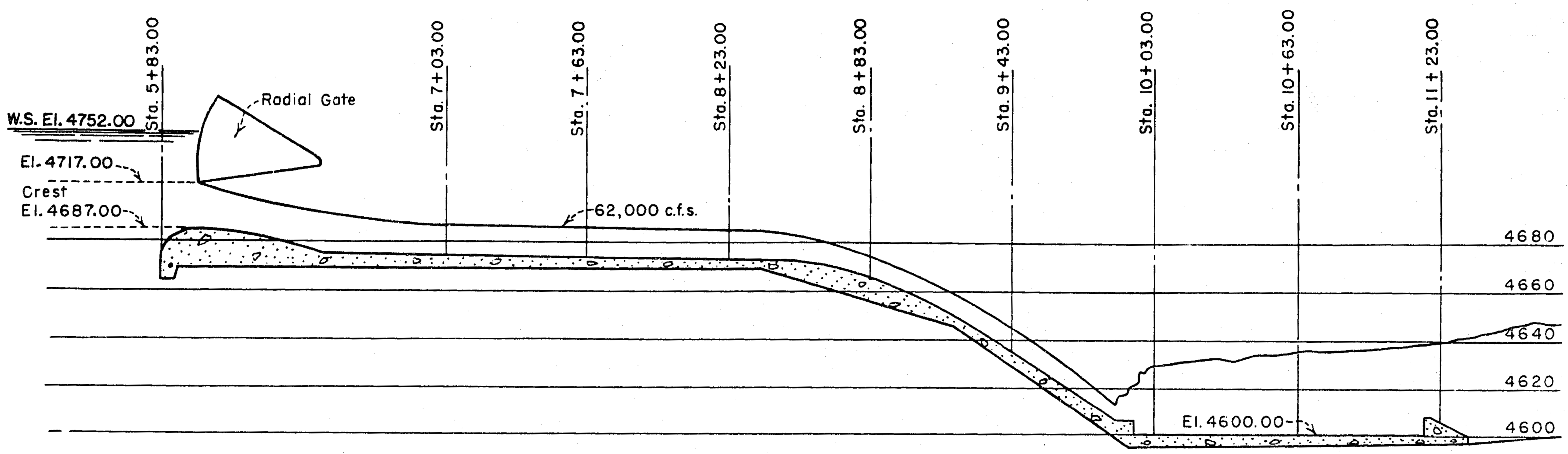


(b) Upstream view.



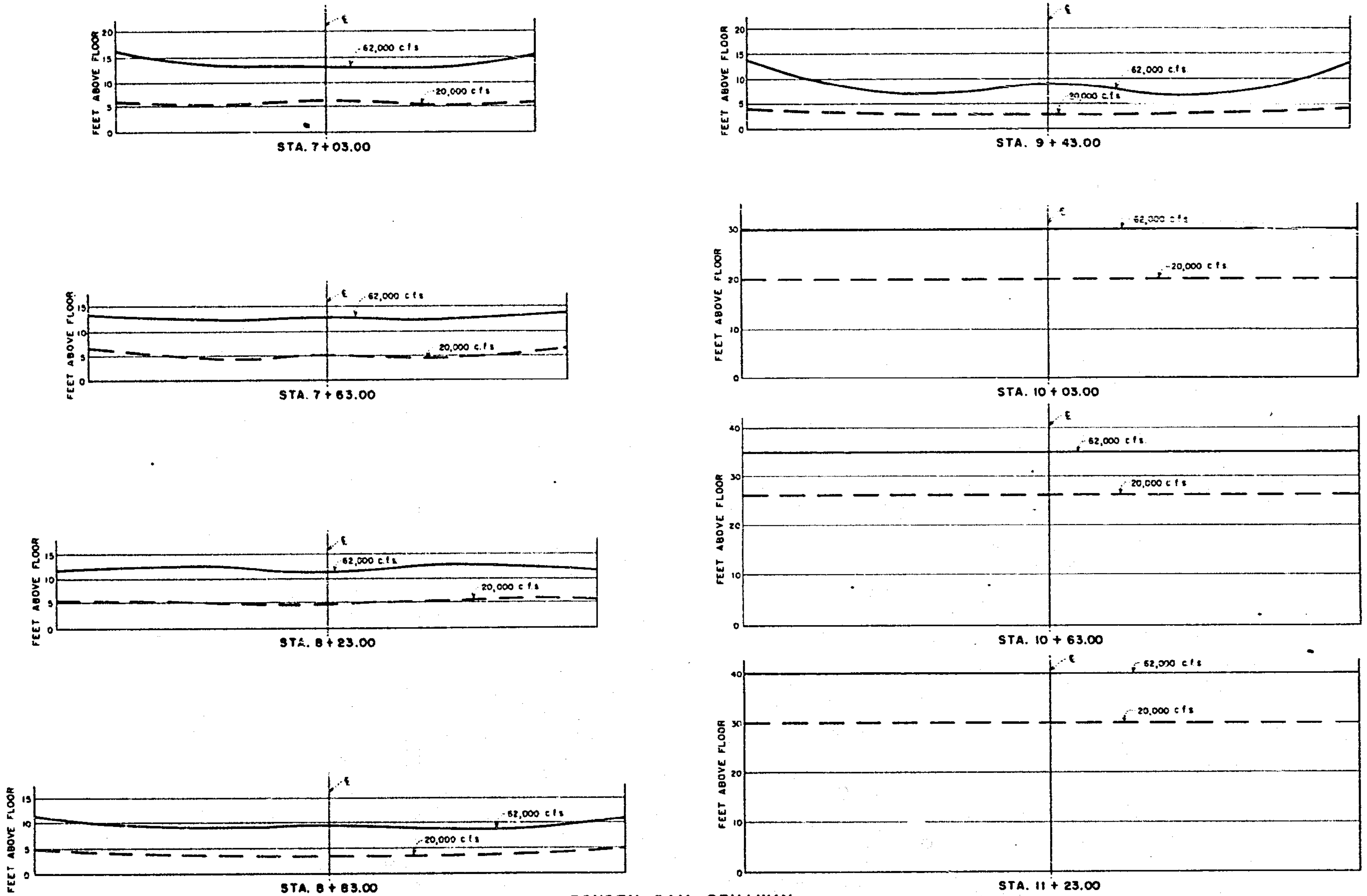
(c) Discharge - 62,000 Second-feet.

BOYSEN DAM SPILLWAY
Model Views - Second Spillway Design - Scheme One
With Training Wall And Gate Pier Modifications
1:60 Model



NOTE: Training wall modification shown in Figure 36(b) and a 6-feet wide center gate pier were used.

BOYSEN DAM SPILLWAY
WATER SURFACE PROFILE ON ℄ OF SPILLWAY
SECOND SPILLWAY DESIGN - SCHEME ONE
1:60 MODEL



BOYSEN DAM SPILLWAY
SECTIONAL WATER SURFACE PROFILES
SECOND SPILLWAY DESIGN - SCHEME ONE
1:60 MODEL

NOTE: Training wall modification shown in Figure 38 (B) and a 6-feet wide center gate pier were used.